Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 FEBRUARY 1959

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1950 LOCH SLOY
1951 FASNAKYLE
1952 LUSSA
1953 GAUR
1954 LUICHART
1955 CLACHAN
1955 QUOICH
1956 INVERGARRY
1956 CEANNACROC
1957 SRON MOR

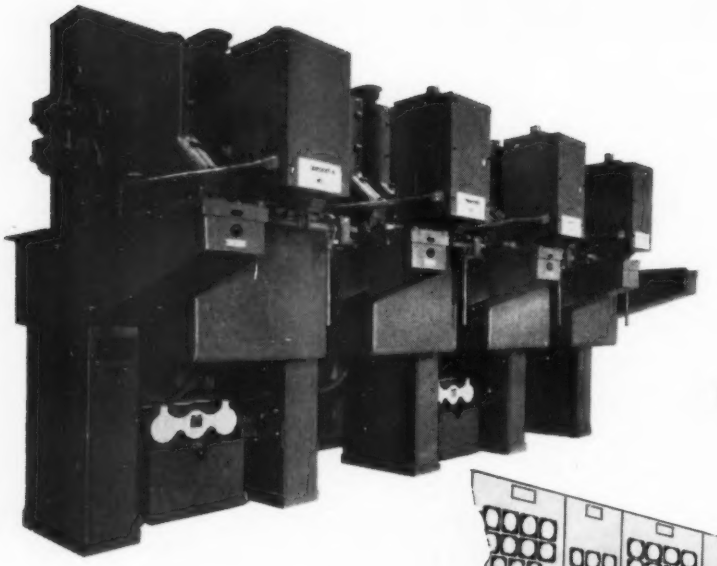
LOCHAY
58 GLENMORISTON

ll more
wer for
e Highlands

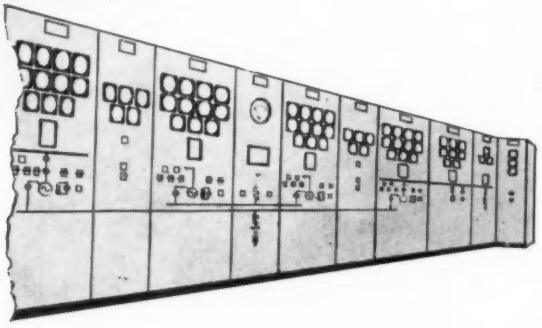
ENGLISH ELECTRIC' 18,000 kW water
ing sets recently commissioned at
nston. The machines were manufac-
at Harland and Wolff's works, Scotstoun.
years, 'ENGLISH ELECTRIC' have installed
and generating equipment with an output
450 kW and water turbines developing
p.

NGLISH ELECTRIC'
kyro-electric equipment

ELECTRIC COMPANY LIMITED, MARCONI HOUSE, STRAND, LONDON, W.C.2.
Hydro-electric Department, Netherton, Bootle, 10
PRESTON · RUGBY · BRADFORD · LIVERPOOL · ACCRINGTON



generator switchgear at Calder Hall



The generators at the Calder Hall Power-station of the Atomic Energy Authority are controlled by Reyrolle 11-kV 500-MVA and 750-MVA switchboards and their associated control-boards

Reyrolle

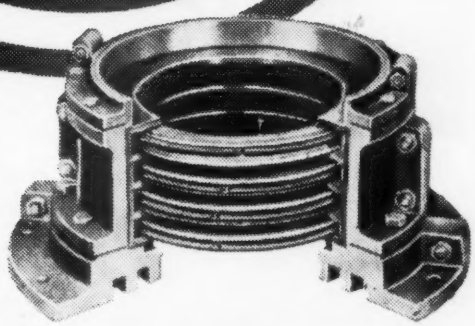
A. REYROLLE & COMPANY LIMITED HEBBURN, COUNTY DURHAM, ENGLAND

The Swedish State Power Board, Stockholm
and Nydqvist & Holm AB, Trollhättan
chose

HUHN carbon ring seals

- *for the three biggest water turbines in the world*

HUHN



at Stornorrfors

- *60 years of experience as specialists in rotating shaft seals*
- *700 Francis and Kaplan turbines equipped with HUHN carbon ring seals*

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Substation steel structures
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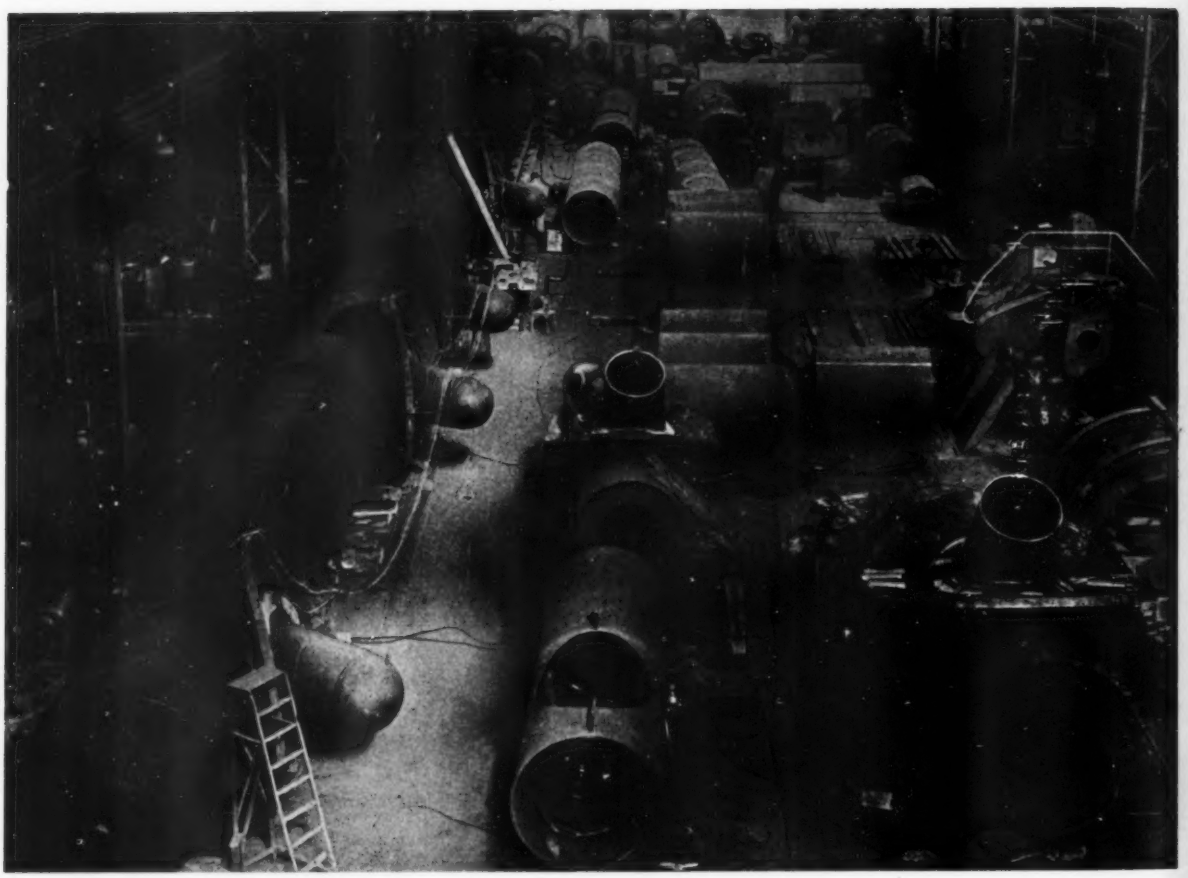
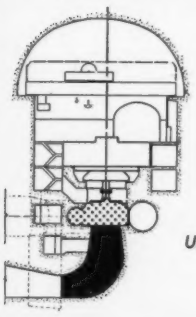


Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

GIOVANOLA

Monthey - Switzerland

KARIBA—ANOTHER BIG STEP TOWARDS COMPLETION



SECTION THROUGH UNDERGROUND POWER HOUSE

The first stay ring for the Boving turbine of No. 1 unit at the Kariba underground power station, was shipped in May 1958. It is shown here, prior to shipment, at the works of Markham & Co. Ltd., Chesterfield, complete with guide apparatus assembled.

Output 140,000 H.P.
Normal speed 167 r.p.m.
Max. nett head 340 ft.

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& COMPANY LIMITED

WATER POWER ENGINEERS

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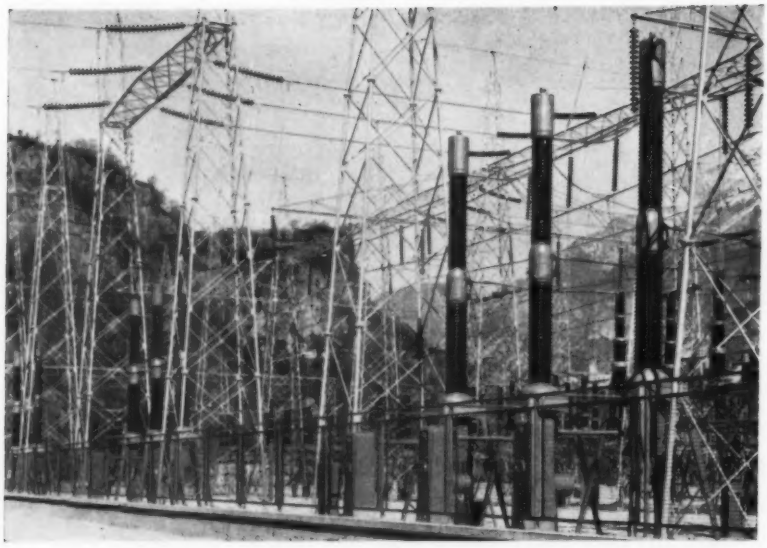
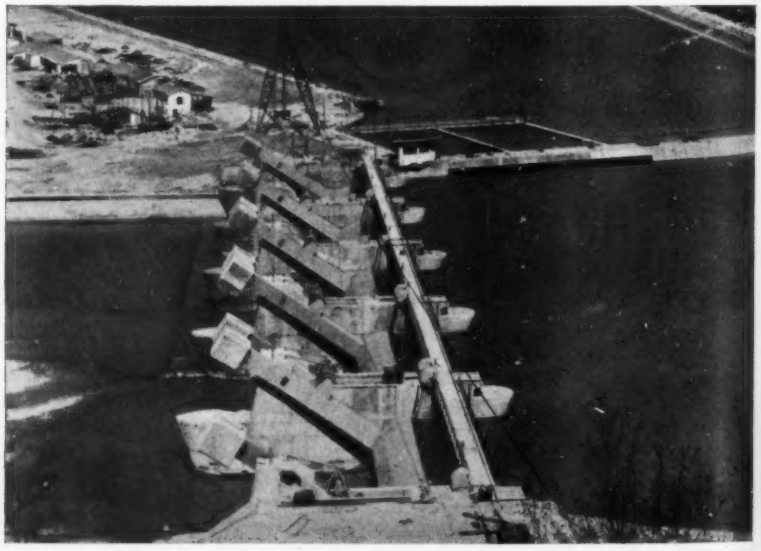
OFFICINE ELETTRO MECCANICHE GALILEO
DI BATTAGLIA TERME S.p.A.



circuit breakers, isolating switches and measuring transformers for 220 kV. Somplago Power station

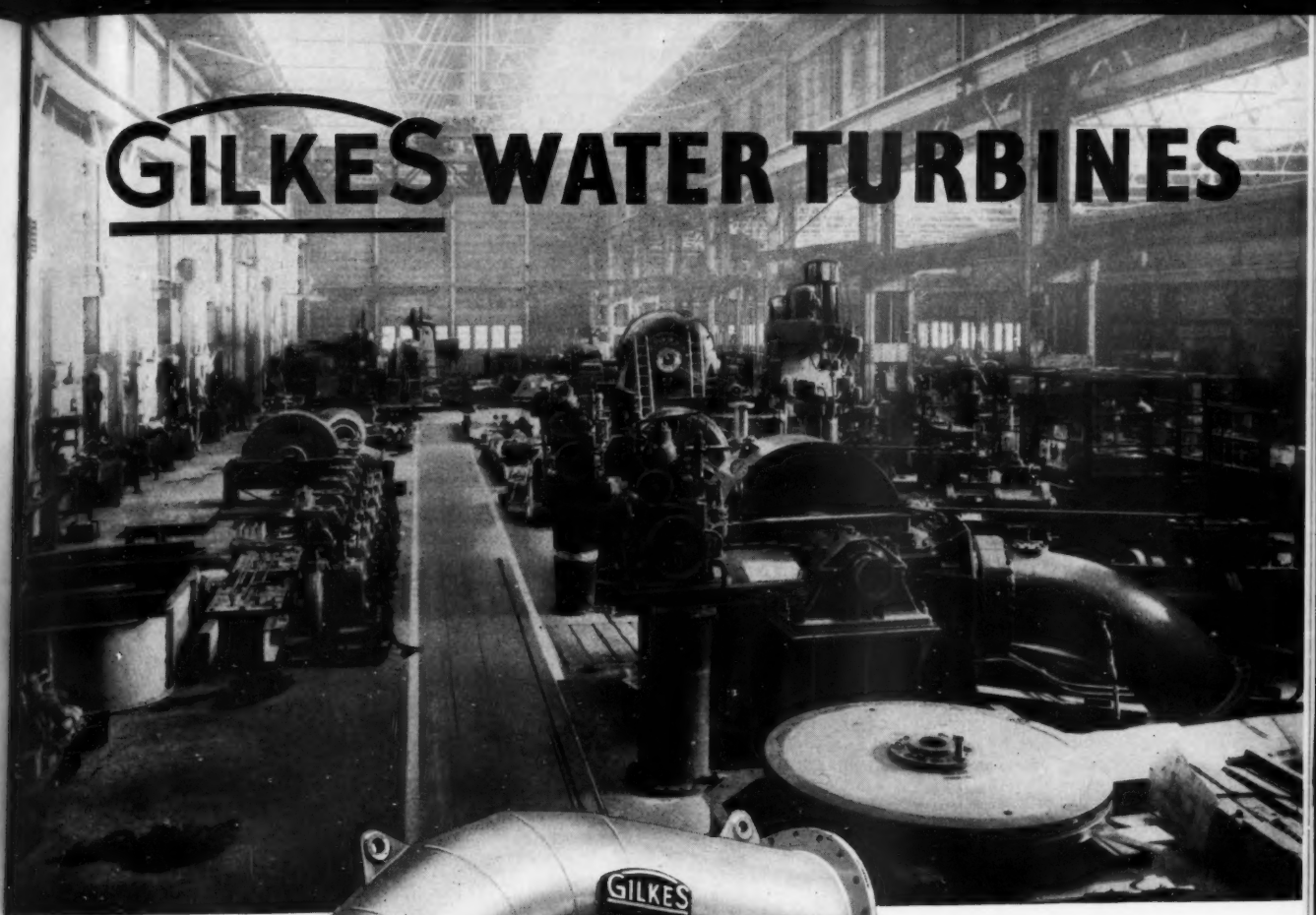
Officine Elettromeccaniche Galileo build electrical equipment for medium, high and ultra-high voltage, travelling cranes, gates of any type, racks, rack rakes and any other metallic frame for hydro-electric plants

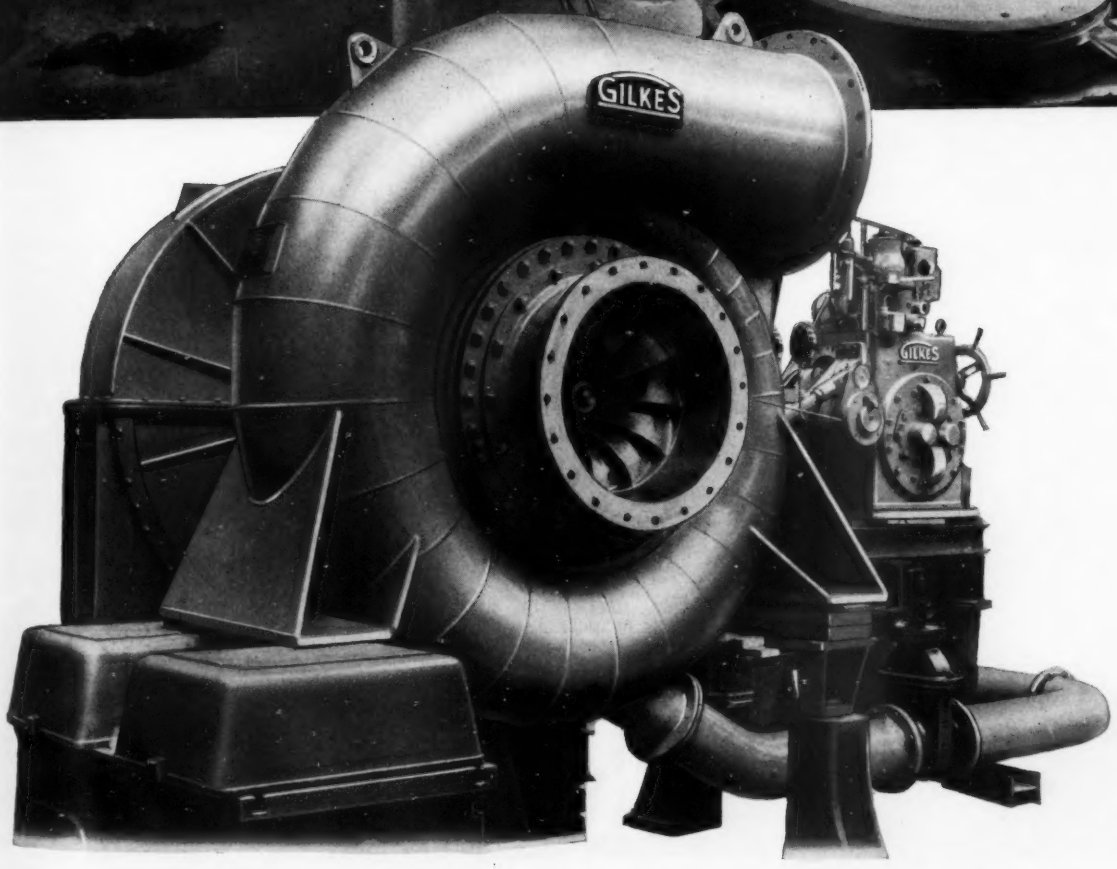


barrage on the Volturno river five automatic sector gates 25 mt. wide; 375 mt. high

For offers and details, please enquire to: Officine Elettromeccaniche Galileo, Servizio Commercio Estero, via Larga, 26, Milano (Italy) — Phone: 877895. Grams: Elettroga, Milano

GILKES WATER TURBINES





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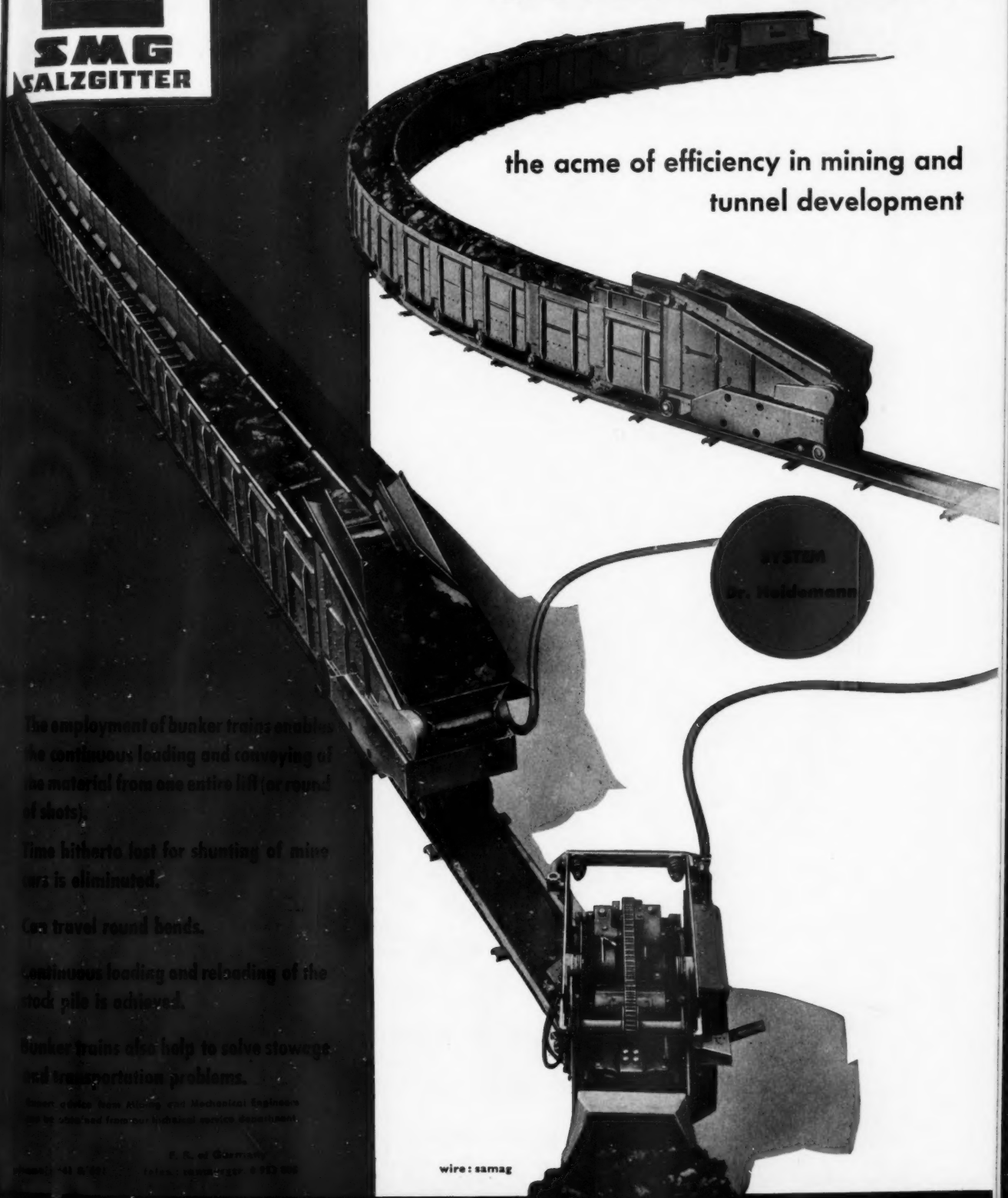

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Time hitherto lost for shunting of mine cars is eliminated.

Cars travel round bends.

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RUNNER FOR THE HIGHEST HEAD
in the world
5,800 FEET

The highest head
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3,700 FEET
is also equipped with CHARMILLES TURBINES
LOS MOLLES POWER PLANT
Two 12,650 HP units

EXCERPT
FROM CHARMILLES
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Installation	Number of units	Net head feet	Unit output HP
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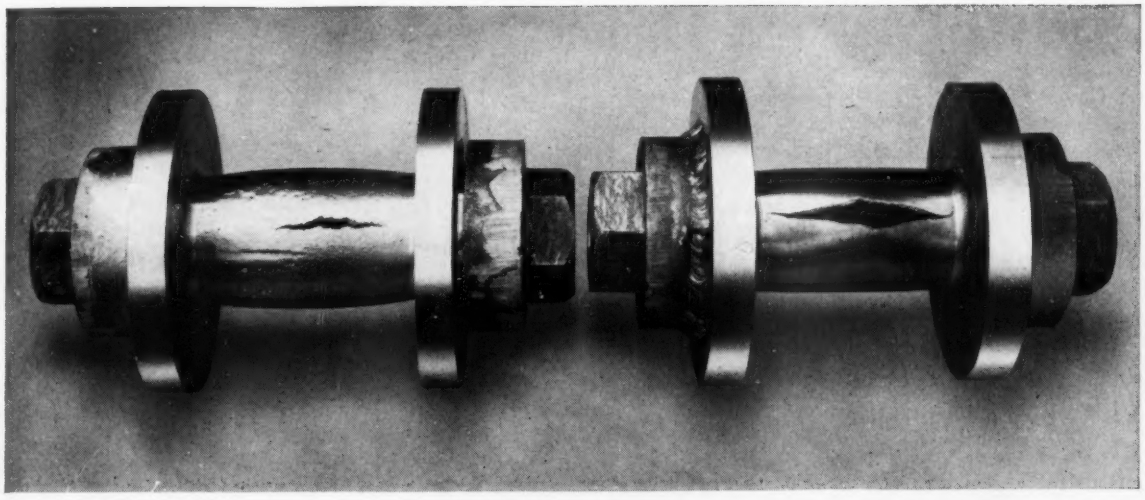
Dixence (Switzerland)	5	5,700	50,000
*Fully (Switzerland)	4	5,400	3,000
Miéville (Switzerland)	3	4,730	47,500
Portillon (France)	1	4,280	53,500

*manufactured in the time of Pic-Pic

No other turbine in the world operates with a head equal to or higher than any of the above

REMARKABLE PERFORMANCE OF ALUMINIUM BRONZE

0·10 IN. SHELL WITHSTANDS OVER 2 TONS PER SQ. IN. !



Recently we tested to destruction two similar hollow castings—one in gun-metal, the other in aluminium-bronze. The gun-metal had a wall thickness of **0·25 in.** and withstood pressures up to 4480 lbs. p.s.i.—a very good performance. The casting in aluminium-bronze burst at a slightly higher pressure (4816 lbs. p.s.i.) **but its wall thickness was only 0·10 in.** Surely an impressive demonstration of the weight/strength characteristics of this alloy.

We were one of the first foundries to cast in aluminium-bronze and to-day we supply castings in this alloy to customers all over the world.

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LEFT *Gun-metal* **0·25 in.** *thick—burst at 4480 lbs. per sq. in.*

RIGHT *Aluminium-bronze* **0·10 in.** *thick—burst at 4816 lbs. per sq. in.*

CASTINGS FROM A FEW OUNCES TO 10 TONS...

in all non-ferrous alloys. Precision machining. Specialists in aluminium-bronze, centrifugal-cast wheel blanks, shell moulded castings and chill-cast rods and tubes. Continuous cast phosphor-bronze bars up to 12 ft. lengths.

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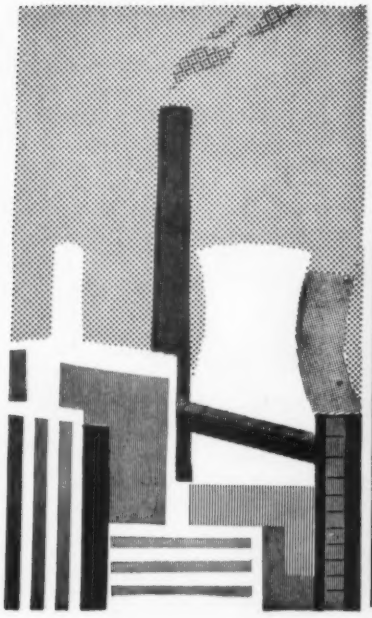
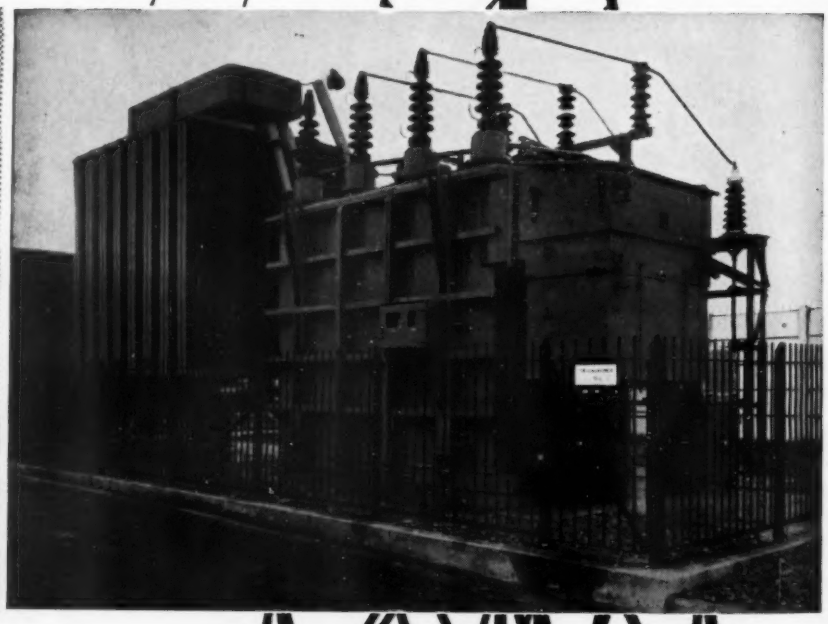
serve the power supplies of Great Britain





YORKSHIRE ELECTRIC TRANSFORMER CO. LTD.
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Tel: Dewsbury 1691-2 Grams: Transforma, Dewsbury

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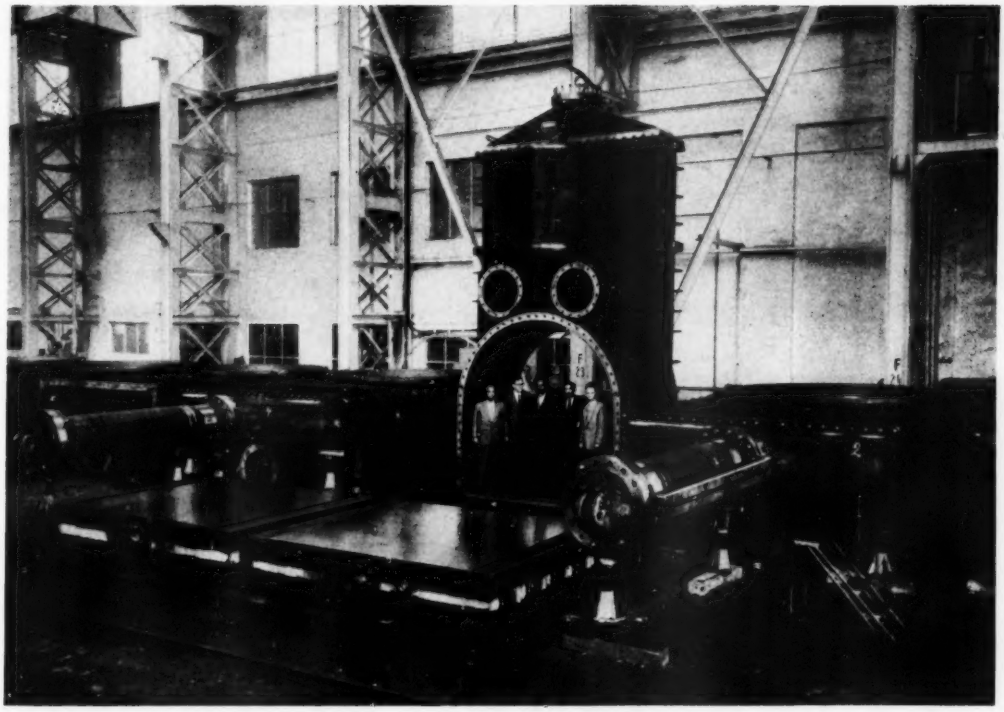

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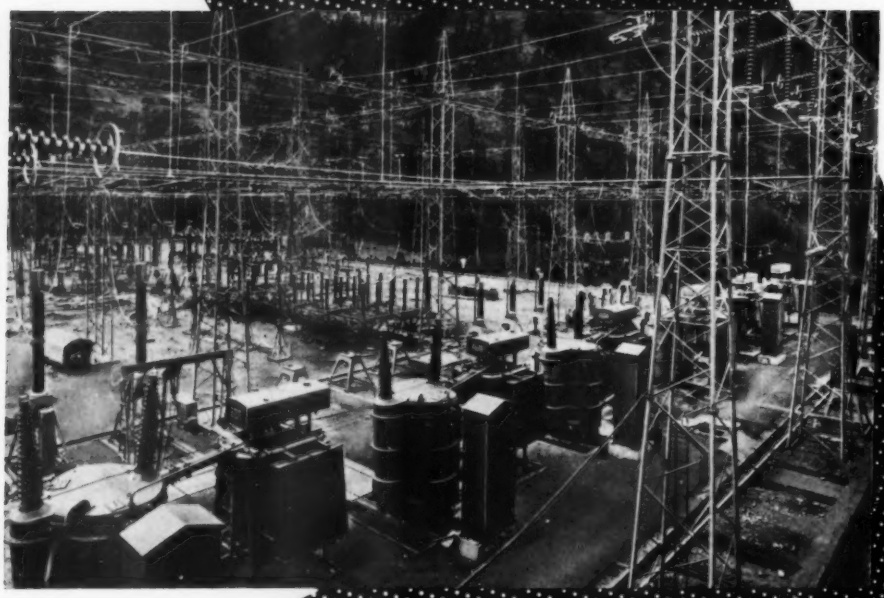

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14 MILES OF TUNNELS

Completed

9 months ahead of schedule

Mitchell Construction completed driving 14½ miles of tunnels on the Killin section of the North of Scotland Hydro-Electric Board's Breadalbane scheme, at Christmas, **more than 9 months ahead of schedule.**

Maximum advance achieved in one week, on one face, was 517 feet. Overall average advance during the nineteen months of driving was ¾ mile a month, with a maximum of 1½ miles a month.

This sustained high speed advance has, it is believed, created new precedents in the Hydro-Electric field.

This success has only been achieved by the careful planning and sense of urgency which Mitchell Construction give to all contracts entrusted to them, a policy which has been successfully applied in driving over 40 miles of tunnels for the board since 1952.

Consulting Engineers: James Williamson & Partners

MITCHELL CONSTRUCTION

building and civil engineering contractors:
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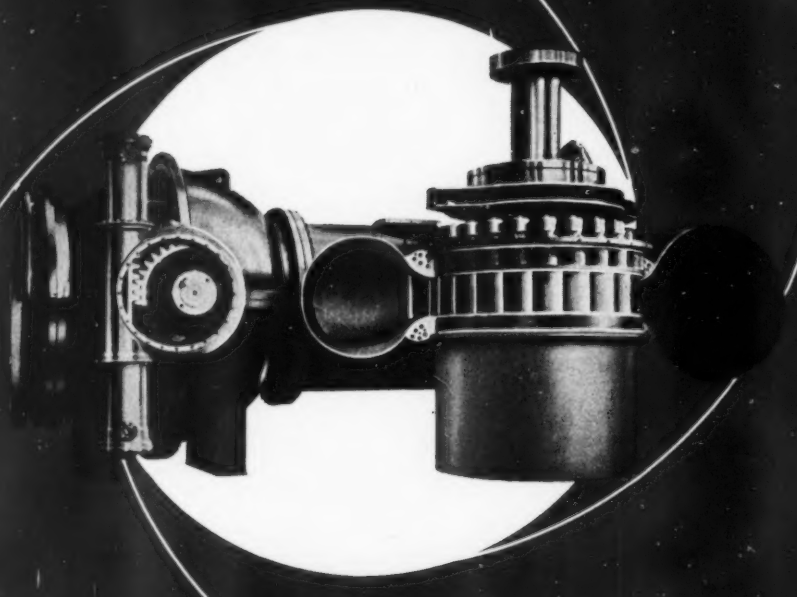
FOR WATER POWER PLANTS



Turbines of the Francis, Kaplan and Pelton type for various heads and outputs, transformers, generators, outdoor and indoor switch-gear, control-rooms, etc.

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BRECO 7½ ton, Standard Single Motor Radial Travelling Cableway, installed in Colombia for the construction of the Anchicaya Dam.

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BRECO—Sole Manufacturers of 'Travelift' Cableways, outside the North American Continent. 'Travelifts' are especially suited to high speed heavy construction work.

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Vertical-Shaft
Hydro Power Alternators

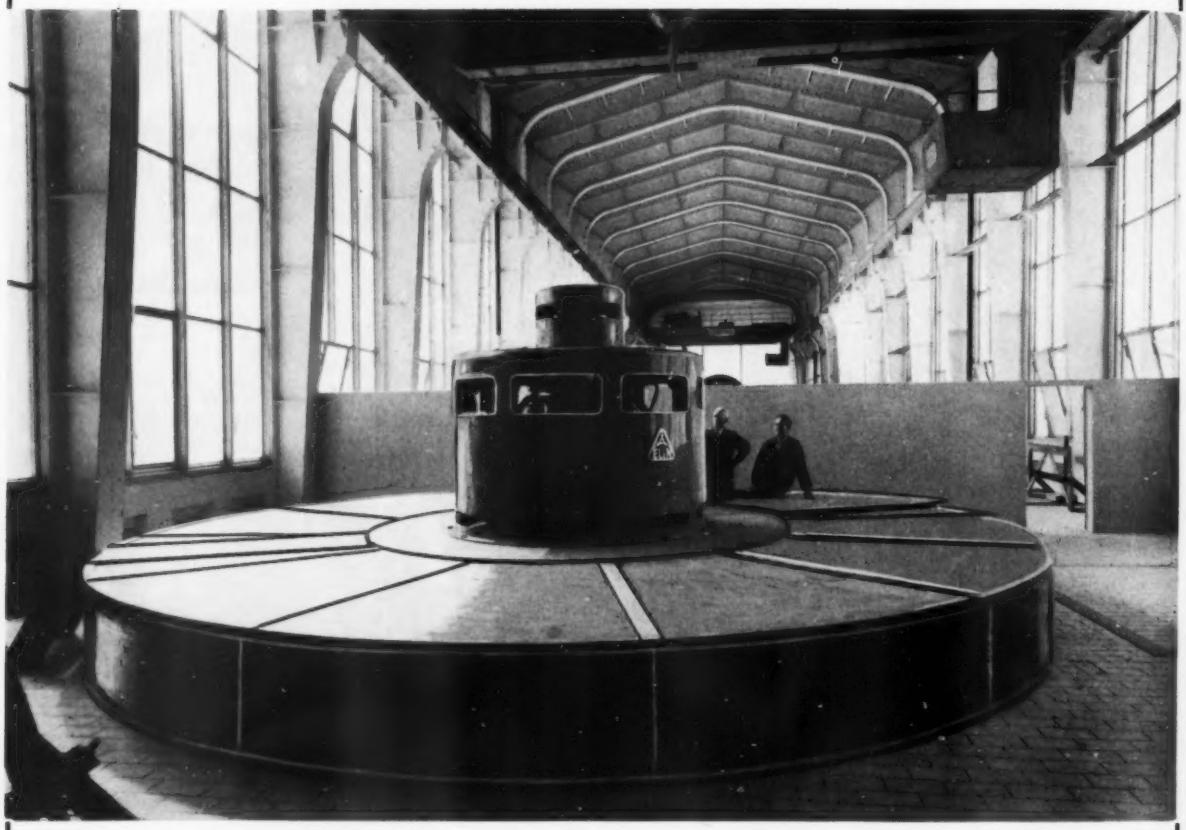

Photo: TKW

A 35,000-kVA vertical-shaft generator, 375 r.p.m., in the power-house of the new Schwarzach Generating Station of Tauernkraftwerke A.G., Salzburg. Four such generators are being supplied by ELIN. The air-to-water heat exchangers of the generator cooling system are arranged for feeding the hot water heating system in the generating station.

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BINGA HYDROELECTRIC PROJECT
FOUR 40,000 H.P.
FRANCIS TURBINES
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COSTRUZIONI MECCANICHE RIVA
VIA STENDHAL 34 MILANO ITALY

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This illustration shows a machine especially built for laboratory use. Our experience in the field of hydraulics has enabled us to extend our manufacturing range to cover small and medium size propeller pumps. We welcome enquiries for axial flow pumps for industrial and agricultural applications, or for research laboratories.

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Penstocks for Hydro-Electric Power Stations

Study, planning, design and erection of complete:

open-laid and trench-laid pipe lines

pipe lines in tunnels

lined pressure shafts, surge tanks, etc.

pipe lines for irrigation and drinking-water supplies

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Branch pipe with Sulzer stiffening collar

Sulzer Brothers Limited,
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Sulzer Bros. (London) Ltd.,
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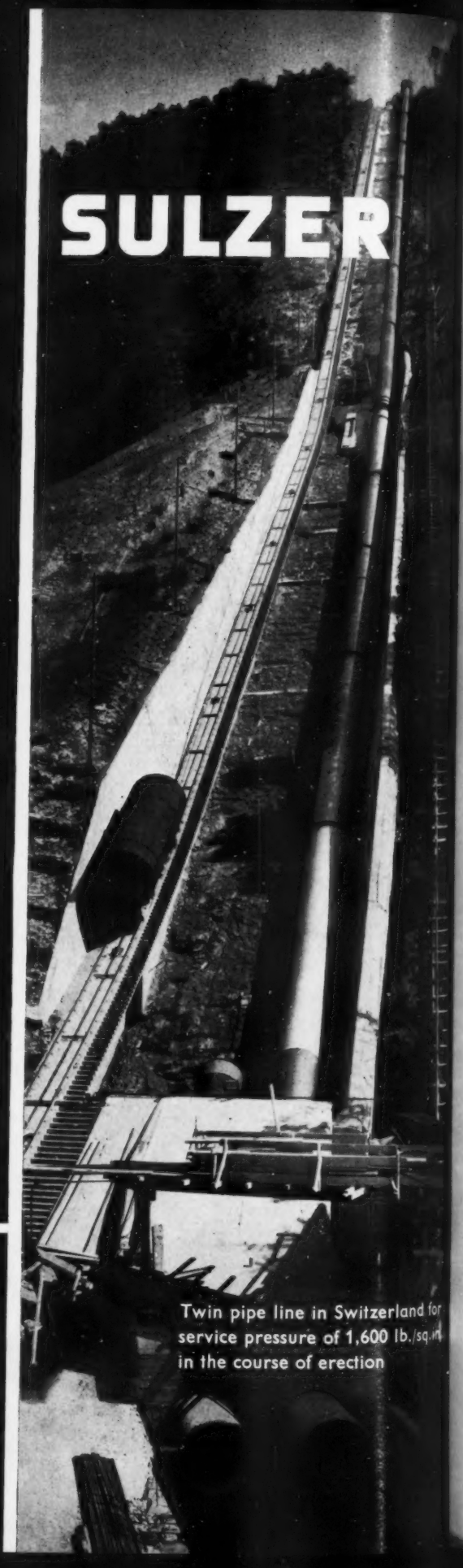


Twin pipe line in Switzerland for service pressure of 1,600 lb./sq.in. in the course of erection

NEW METHOD OF RAISE-DRIVING

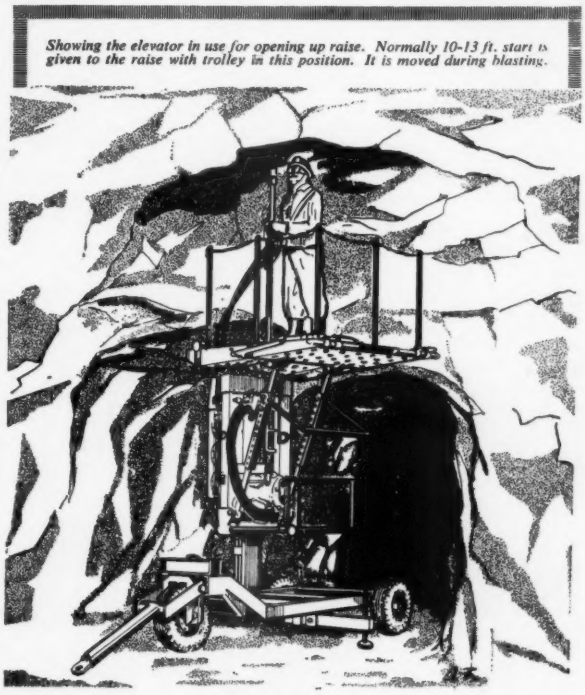
ACE Alimak Drilling Platform reduces hazards – increases output

An entirely new method of driving raises which is destined to replace present conventional methods has just been introduced for the first time into the United Kingdom.

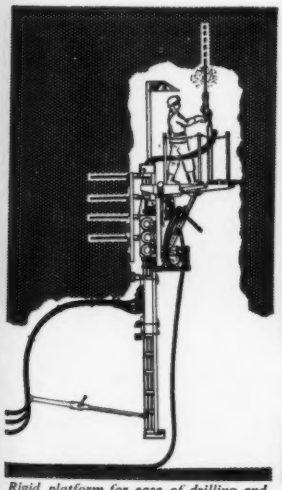
Extensive tests carried out in Sweden over the past twelve months have proved that not only is this new method faster in all types of rock, but that much of the heaviest and most dangerous work associated with existing methods has been eliminated.

Principle of operation

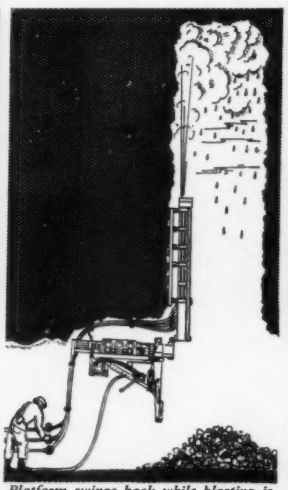
Basically, the equipment consists of an easily transportable all steel elevator, the platform of which is equipped with a compressed air motor enabling it to climb a rack equipped guide rail. The operator is raised to the working face where he drills for charges. After descending, the platform is folded back for blasting, washing out and ventillating. The operator then again ascends on the platform and fits another 3 ft. or 6 ft. rail section; he is now ready to recommence the cycle. All these operations are carried out by one man, all services being carried in the rail section—air, water, electricity and telephone. The platform is also equipped to carry all necessary drilling tools.



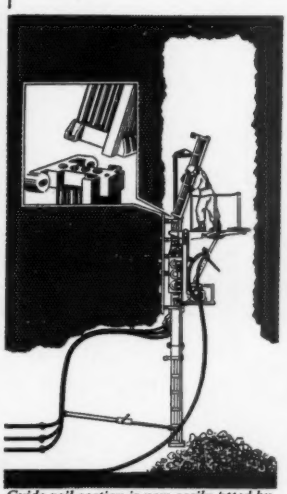
Showing the elevator in use for opening up raise. Normally 10-13 ft. start is given to the raise with trolley in this position. It is moved during blasting.



Rigid platform for ease of drilling and placing charges.



Platform swings back while blasting is carried out. Note air and water mixture to remove all traces of explosion gases.



Guide rail section is now easily fitted by one man for continuing raise.

The ACE Alimak Drilling Platform can be used for any type or angle of raise, and is not confined to vertical shafts. Its design incorporates many safety factors which greatly reduce the hazards with which operatives are normally faced in present raise-driving methods.

Introduced by A.C.E. Machinery Ltd. *Full details and specification on request.*

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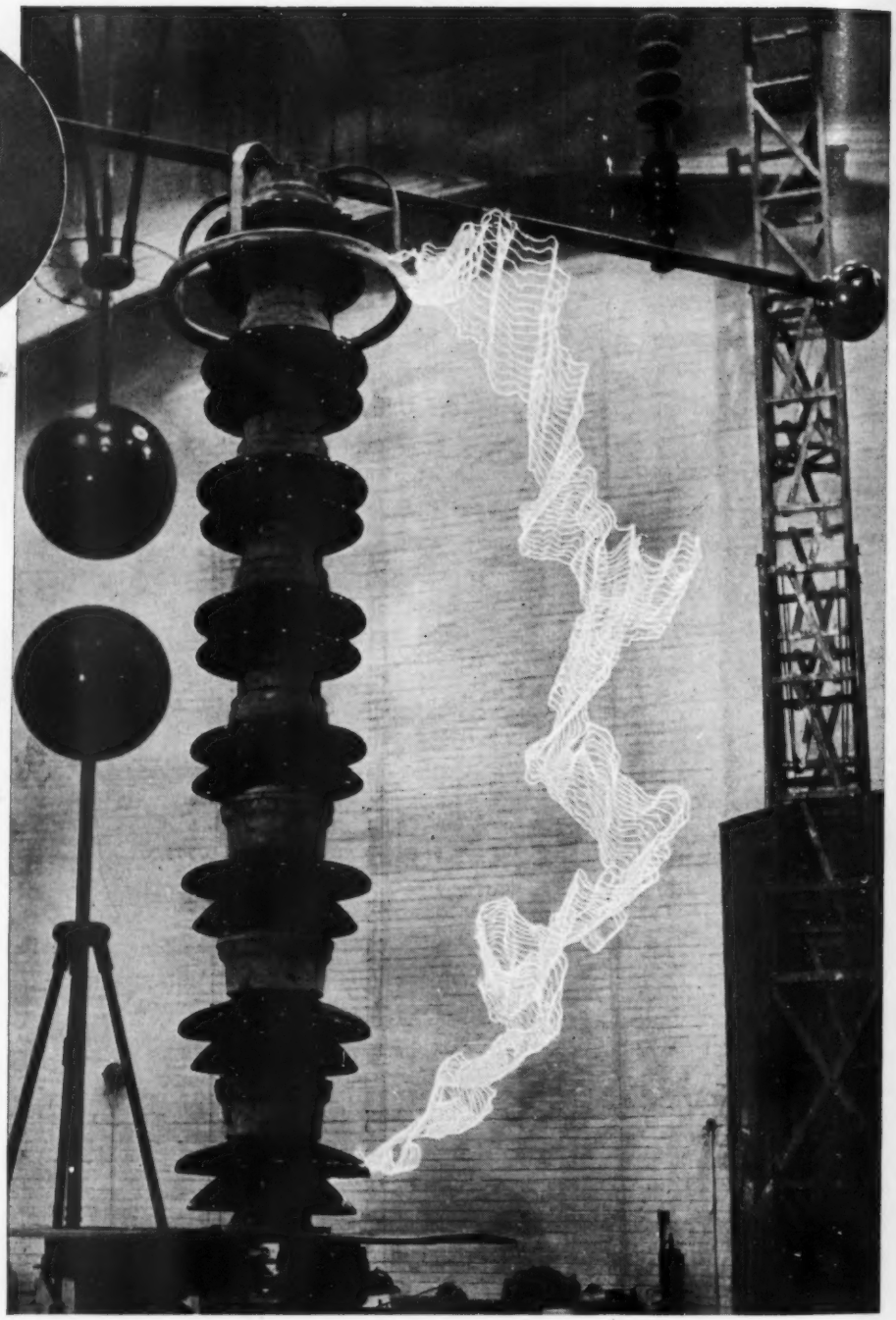
ACE

TRADE MARK

ACE 80

275kV. post insulators for the British Super Grid



One of the S.P.P. 8-unit post insulators for the B.E.A. 275 kV Super Grid undergoing the 50-cycle flashover test at 705 kV during the official type tests carried out at the million-volt laboratory at Stourport-on-Severn.

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Stourport-on-Severn, Worcestershire. Telephone : Stourport 2271 Telegrams : Steatain, Stourport

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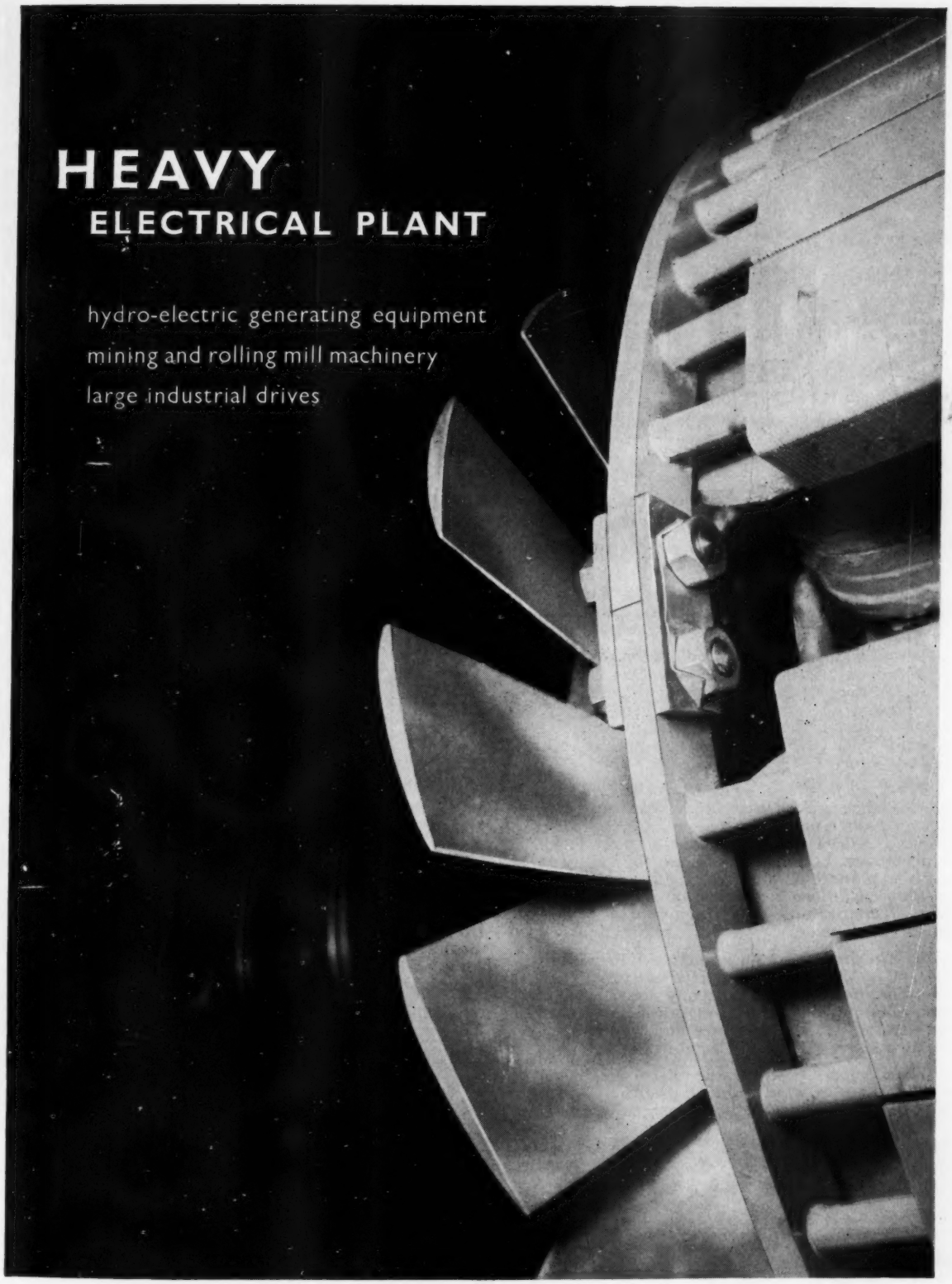

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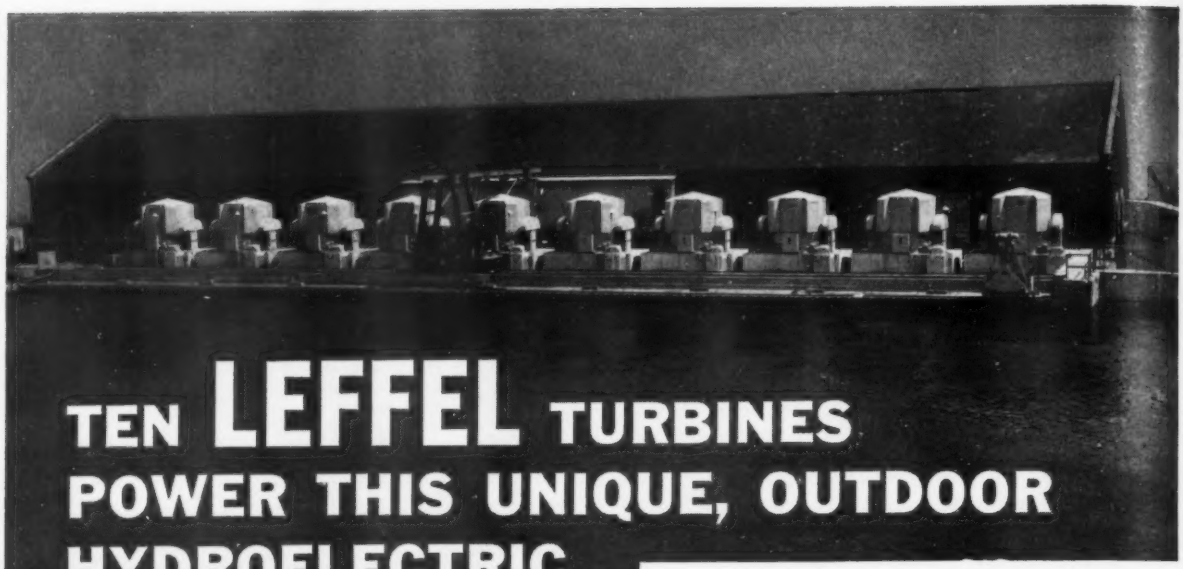


TEN LEFFEL TURBINES POWER THIS UNIQUE, OUTDOOR HYDROELECTRIC PLANT

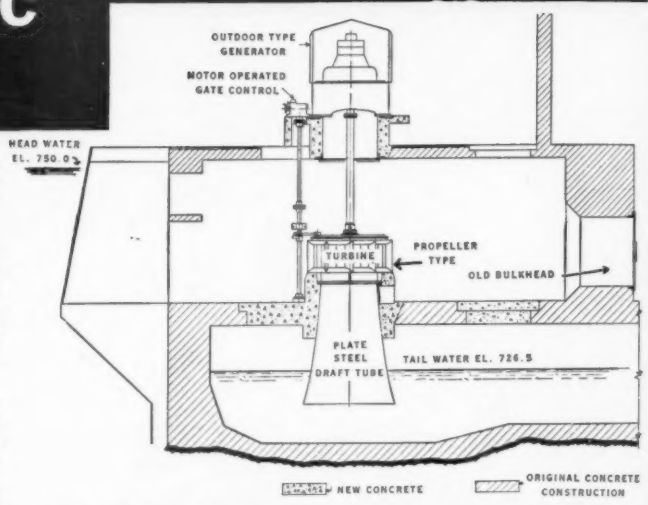
Another installation of efficient Leffel turbines is at the Lower Dam Hydro Plant, St. Anthony Falls, of the Northern States Power Company.

This plant is located a half-mile below the Falls of St. Anthony on the Mississippi River at Minneapolis, Minnesota. It was reconstructed in 1952 in cooperation with the Engineers Corps, Department of the Army's plan to extend the nine-foot navigation channel in the Mississippi River at Minneapolis.

The most feasible method for rehabilitation of the old plant was the installation of vertical, single runner units centered on the old multiple runner turbine settings outside the existing powerhouse. Unique steel housings, known as "Igloos", were used to protect the generator units from adverse weather conditions frequent in this upper Midwest area (Temperatures range from 105 to −30 degrees Fahrenheit).

By utilizing the existing structure in the rehabilitation of the Lower Dam Hydroelectric Plant, Northern States Power Company derived a modern, efficient plant at the lowest possible cost. Such savings are typical whenever and wherever efficient, creative engineering is put to work.

Leffel engineers have a full grasp of the theories and principles involved with *practical* hydroelectric power development such as this. And



Each of the 10 Leffel hydraulic turbines in the Lower Dam Hydro Plant is rated to develop 1160 H. P. under 23.5 Ft. net head, speed 225 R.P.M.

Leffel, with almost a century of experience, has the technical skill and production facilities necessary to produce turbines and related equipment of the highest quality, durability and efficiency.

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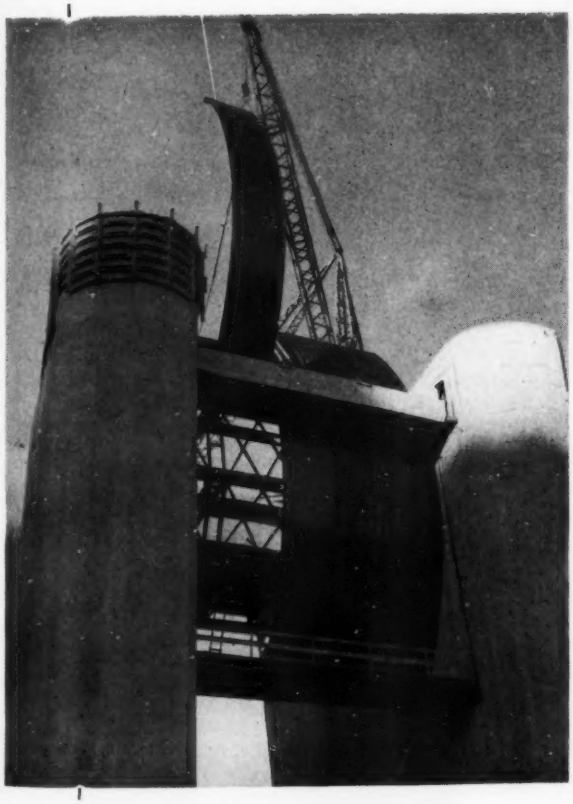

CAP DE LONG: Self-hooped penstock for the Pragnères Power Station, in the Pyrénées.
Head 1,252 metres, Flow 14,6 cu.m./sec.
Diameter 2.25 to 1.60 metres, Power 200,000 H.P.

GRANGENT DAM on the Loire
One of the four Crest Gates, Sector Gate Type (12·5 m. x 14·4 m.) during erection.

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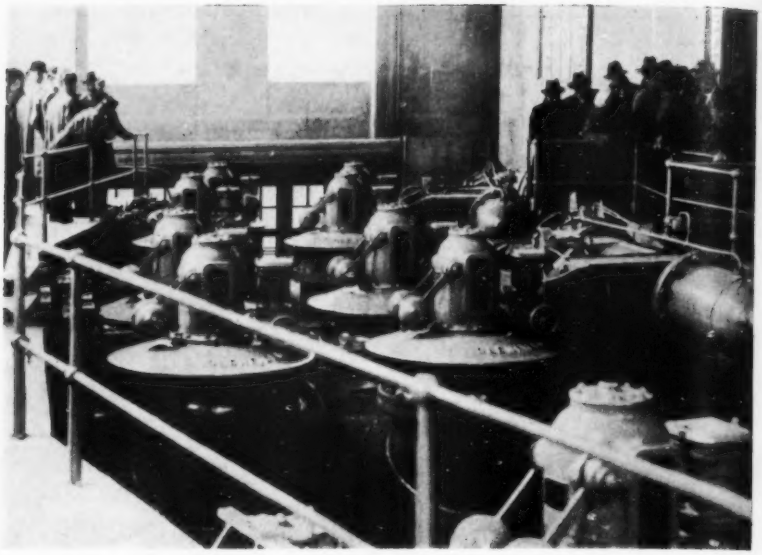
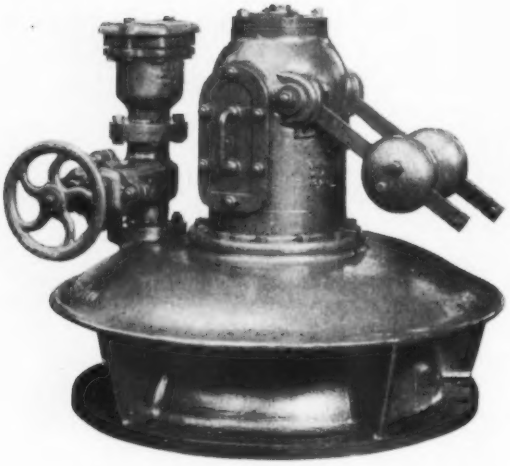


GLENFIELD VALVES
for Pipelines



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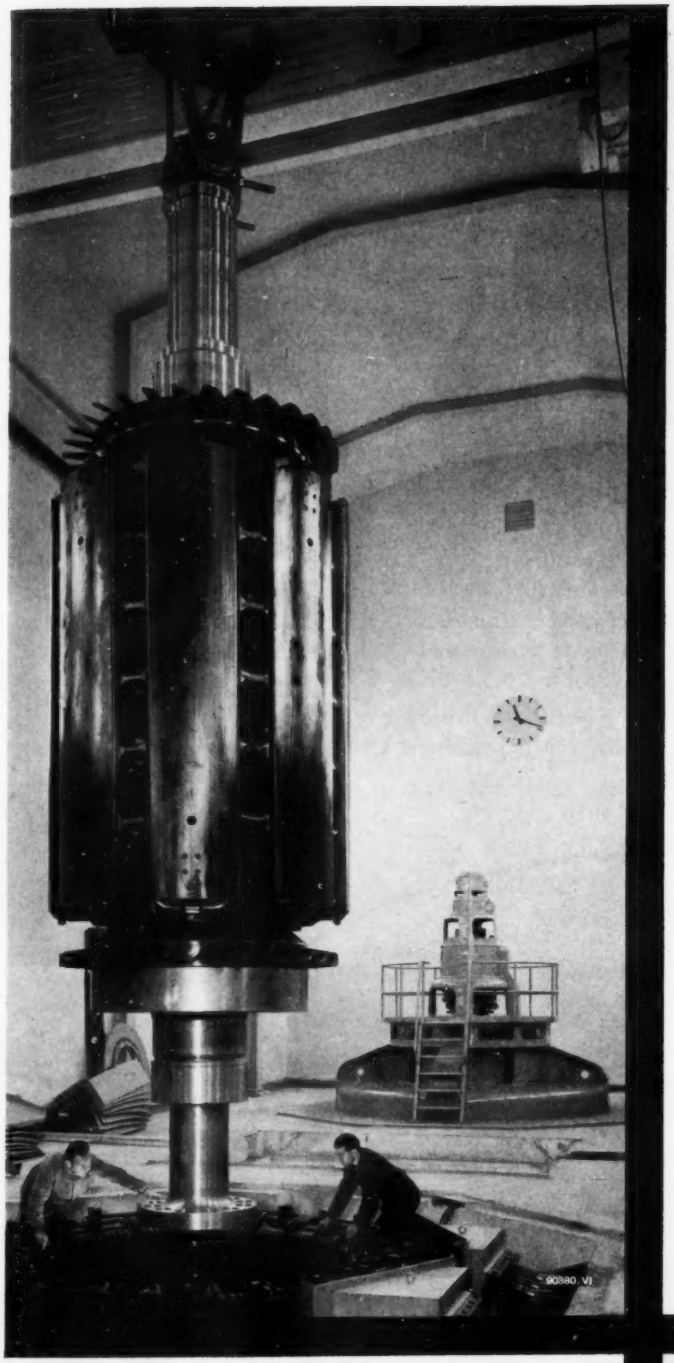

Brown Boveri leaders in the design of high-speed salient-pole waterwheel generators

◀ **Lowering the rotor into the second alternator at the Fionnay Power Plant in Switzerland, owned by the Forces Motrices de Mauvoisin, S.A., Sion, where three generators rated 60 MVA, 0.8 p.f., 750 r.p.m., 10.5 kV, 50 c/s are being installed.**

Some high-speed waterwheel generators at present under construction, or recently installed:

No. of Units	Output kVA	Shaft	Speed r.p.m.	Power Station	Country
1	23 000*	V	1 000	Oberaar	Switzerland
2	20 000	H	1 000	Ernen	Switzerland
3	60 000	V	750	Fionnay	Switzerland
3	29 000	H	750	Motec	Switzerland
1	30 000	V	750	Piottino	Switzerland
3	57 500	V	720/600	Tingambato	Mexico
1	60 000	V	600	Paradela	Portugal
2	45 000	V	600	Hemsil II	Norway
3	17 650	V	600	Neriamangalam	India

* Synchronous motor

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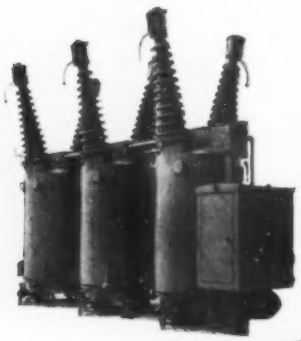
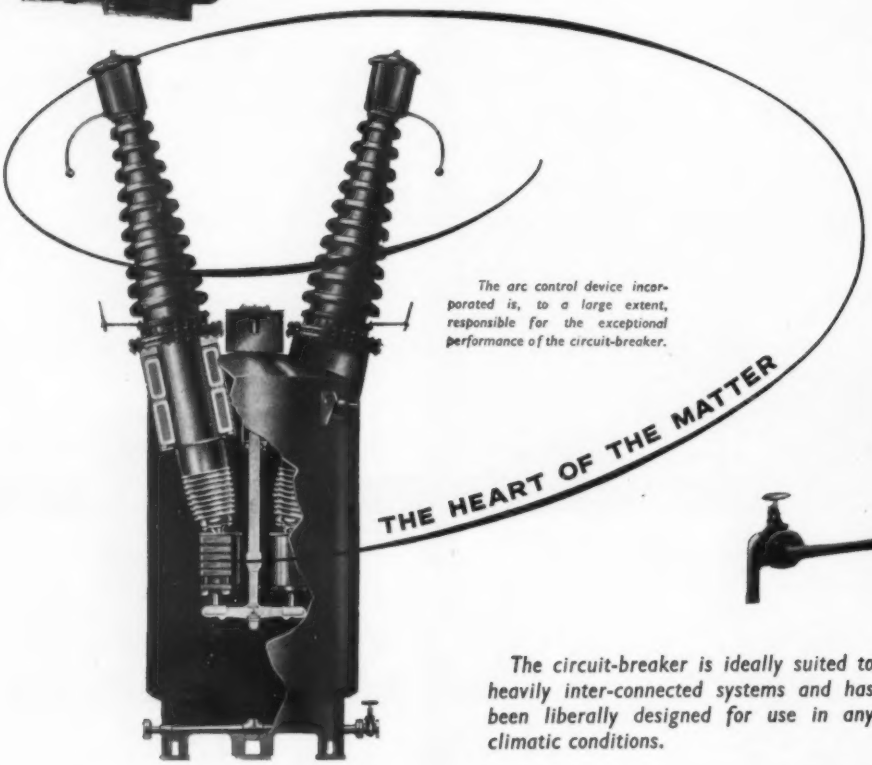


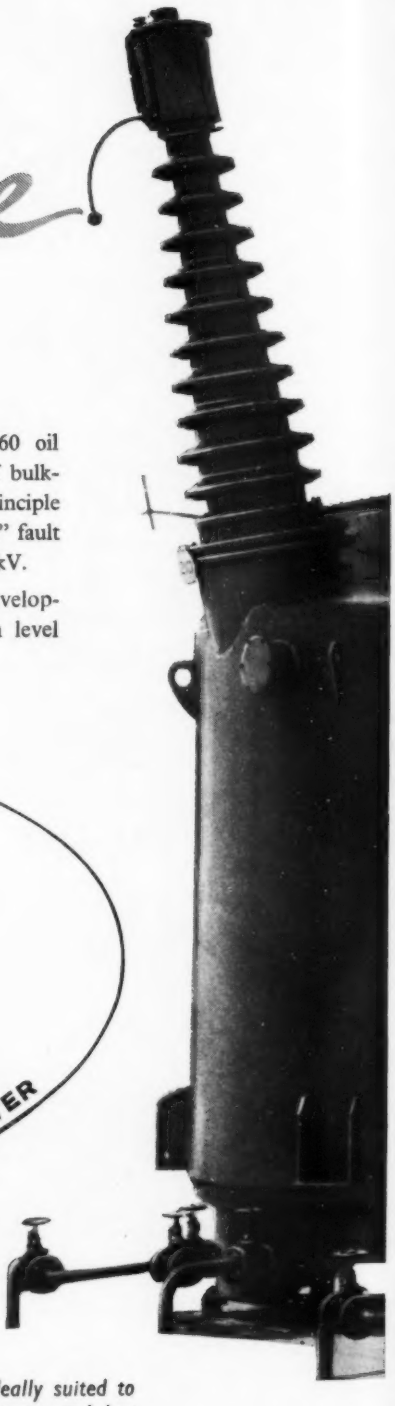
The "FERGUSON PAILIN" Type XOPR60 oil circuit-breaker is outstanding in the field of bulk-oil circuit-breaker design, inasmuch as its principle feature is a *proved* ability to interrupt and "make" fault currents equivalent to 5 000 MVA at 132kV.

In line with "FP" activities in switchgear development generally, this represents a high level of achievement.



The arc control device incorporated is, to a large extent, responsible for the exceptional performance of the circuit-breaker.

THE HEART OF THE MATTER



The circuit-breaker is ideally suited to heavily inter-connected systems and has been liberally designed for use in any climatic conditions.

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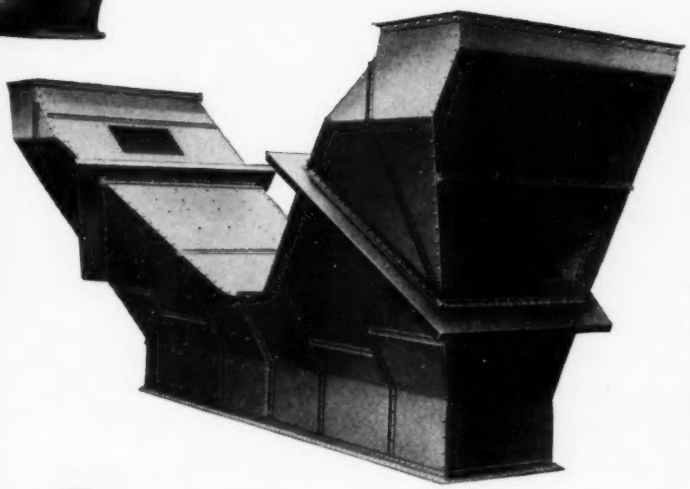

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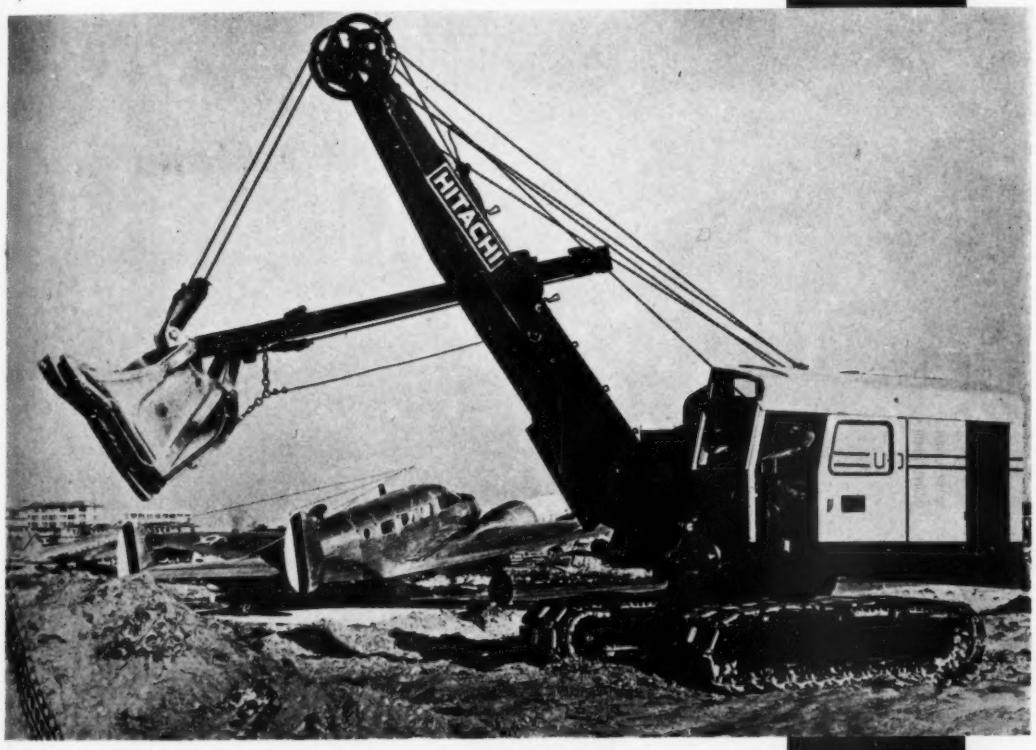

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Hitachi Power Shovel UO6 working at Rio de Janeiro Airport

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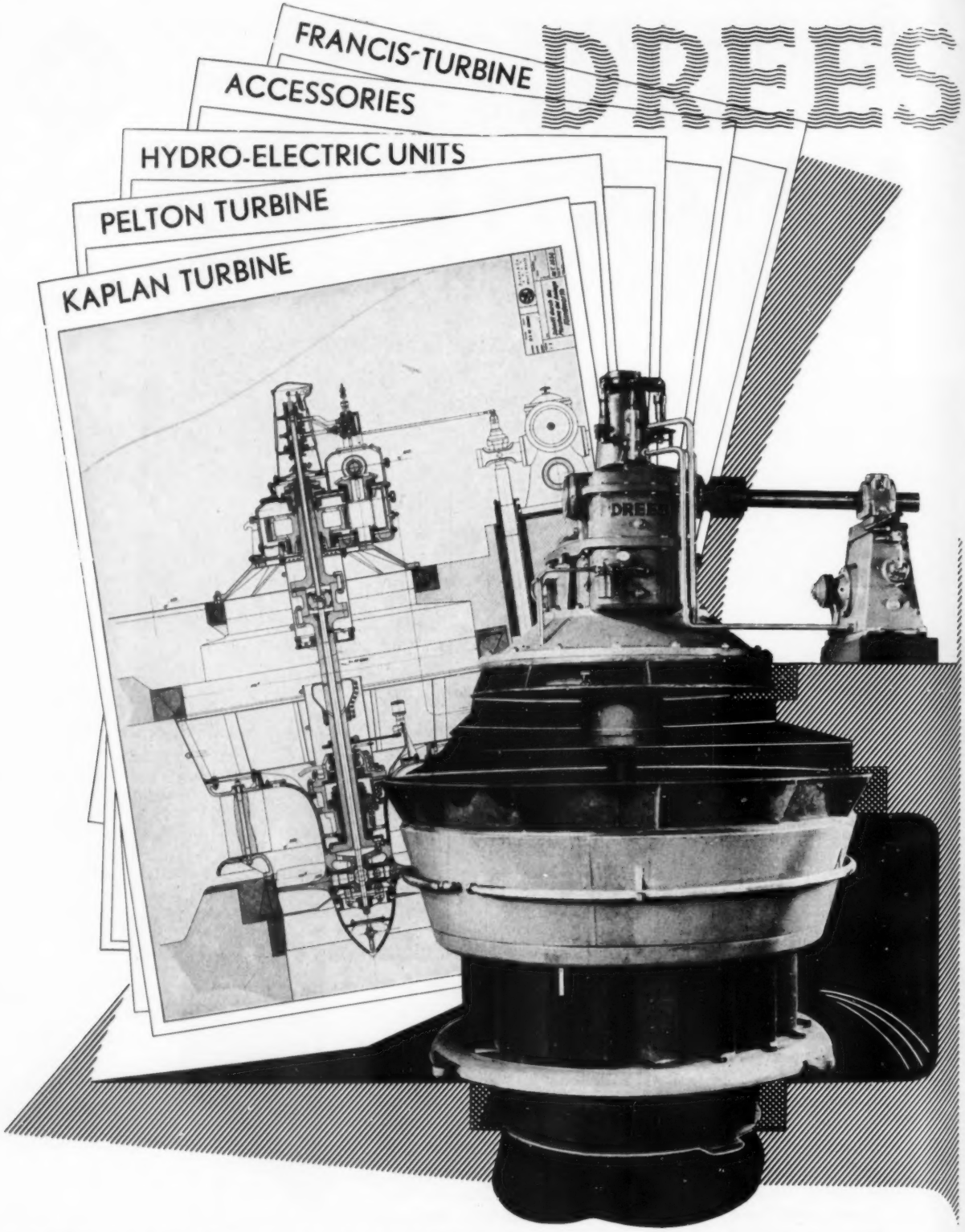

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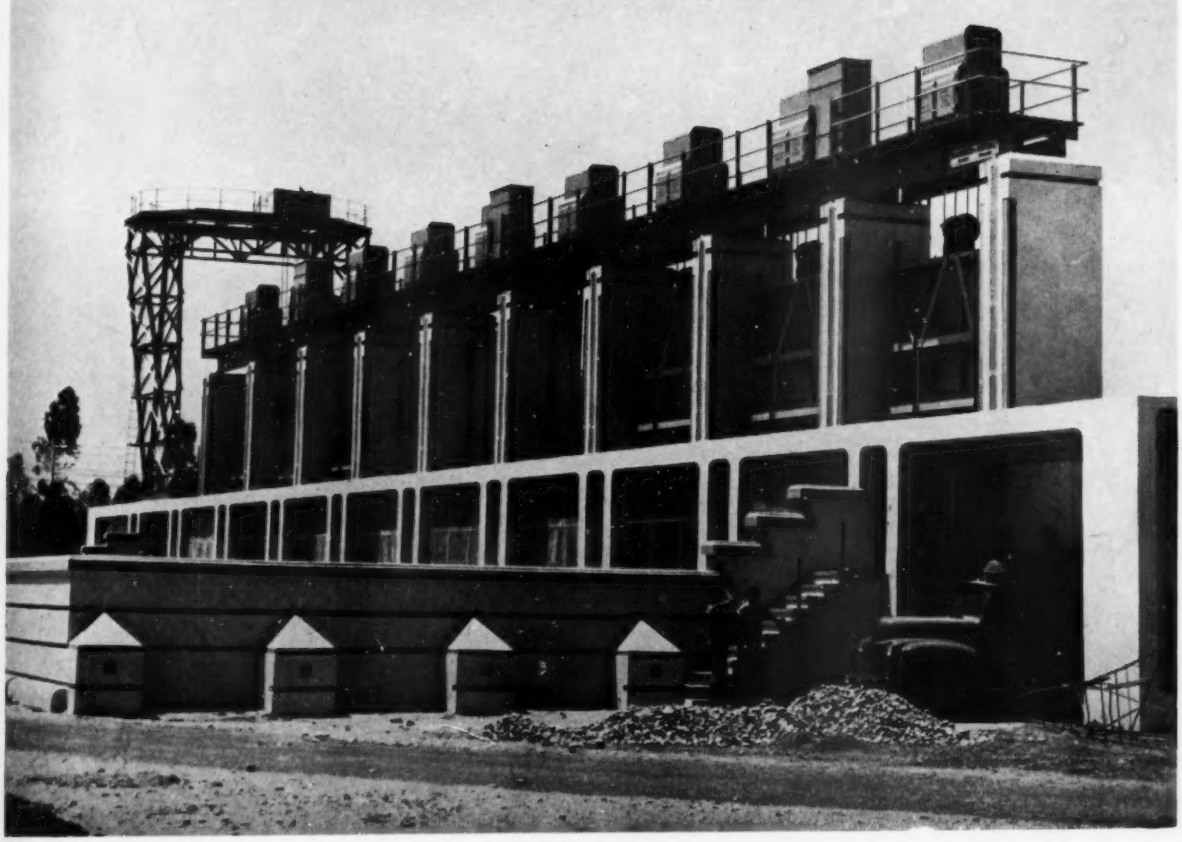

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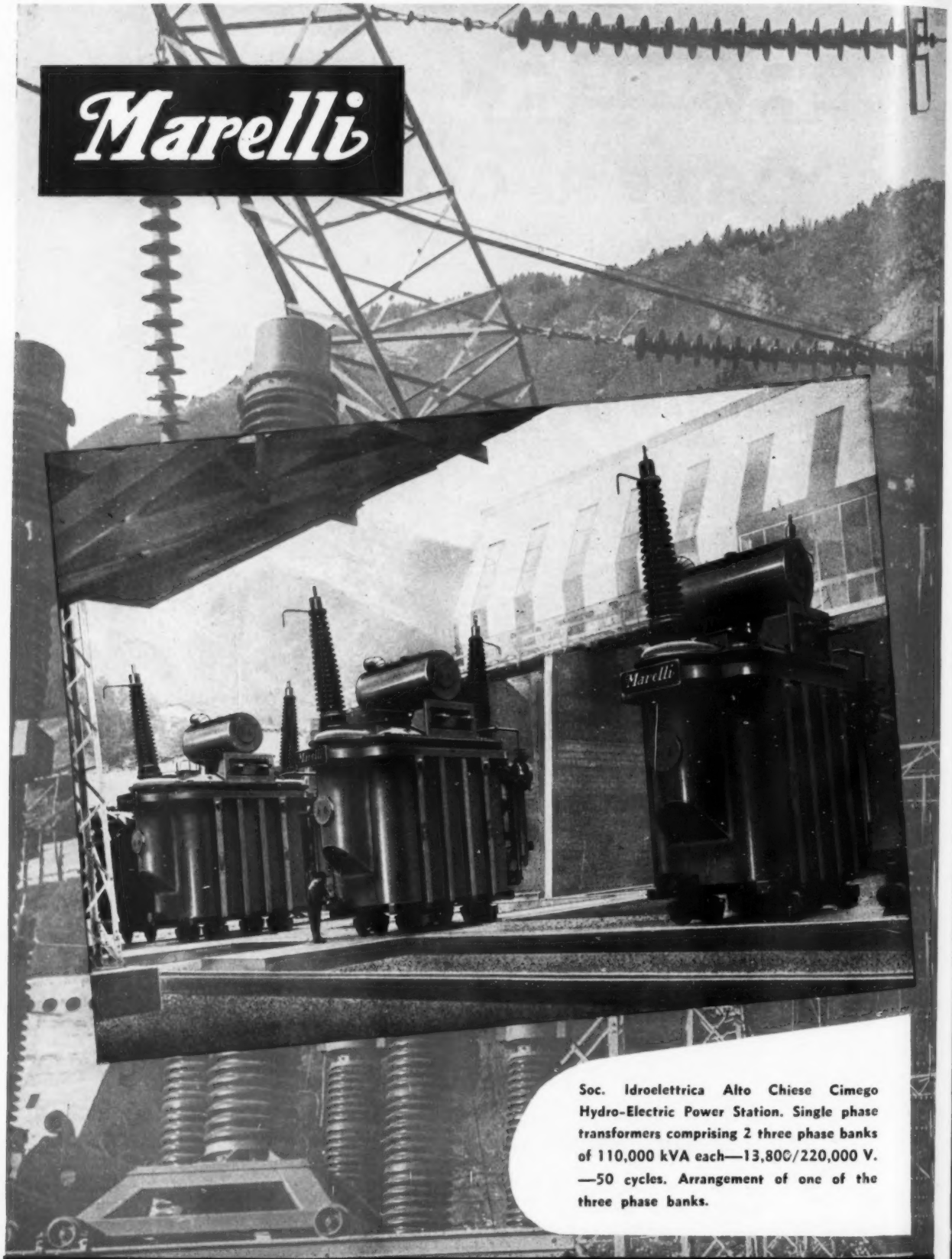

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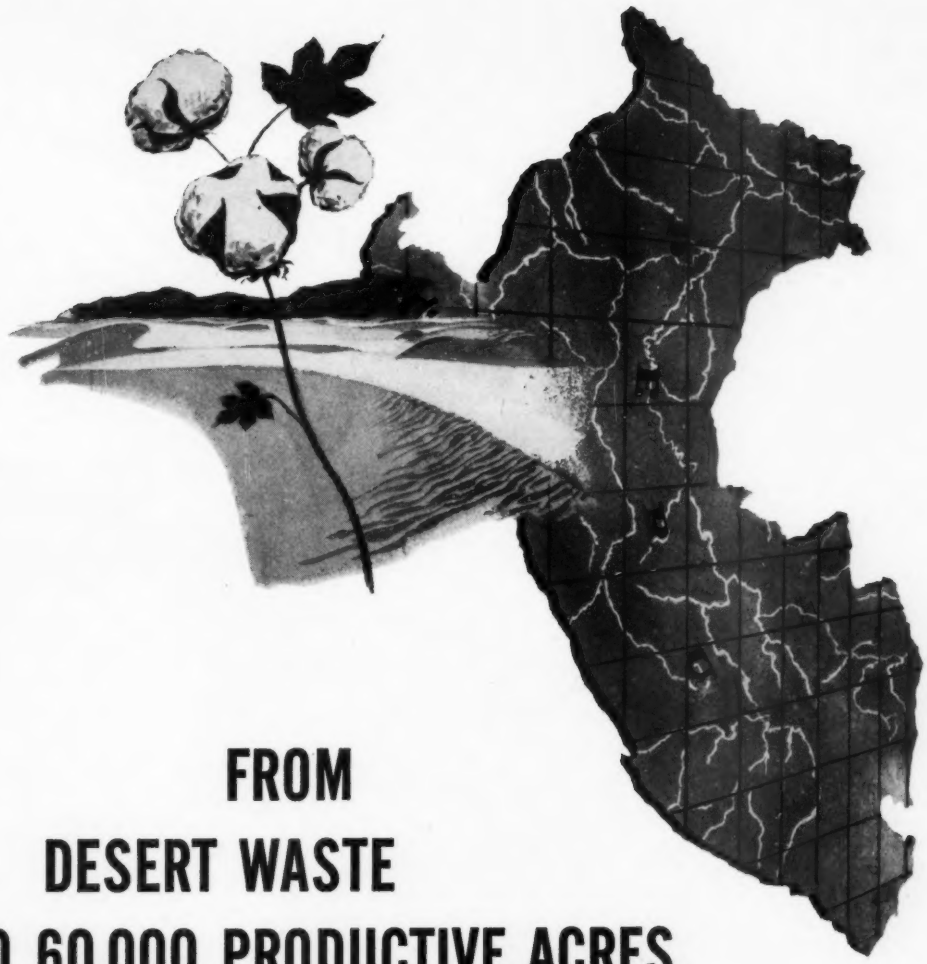

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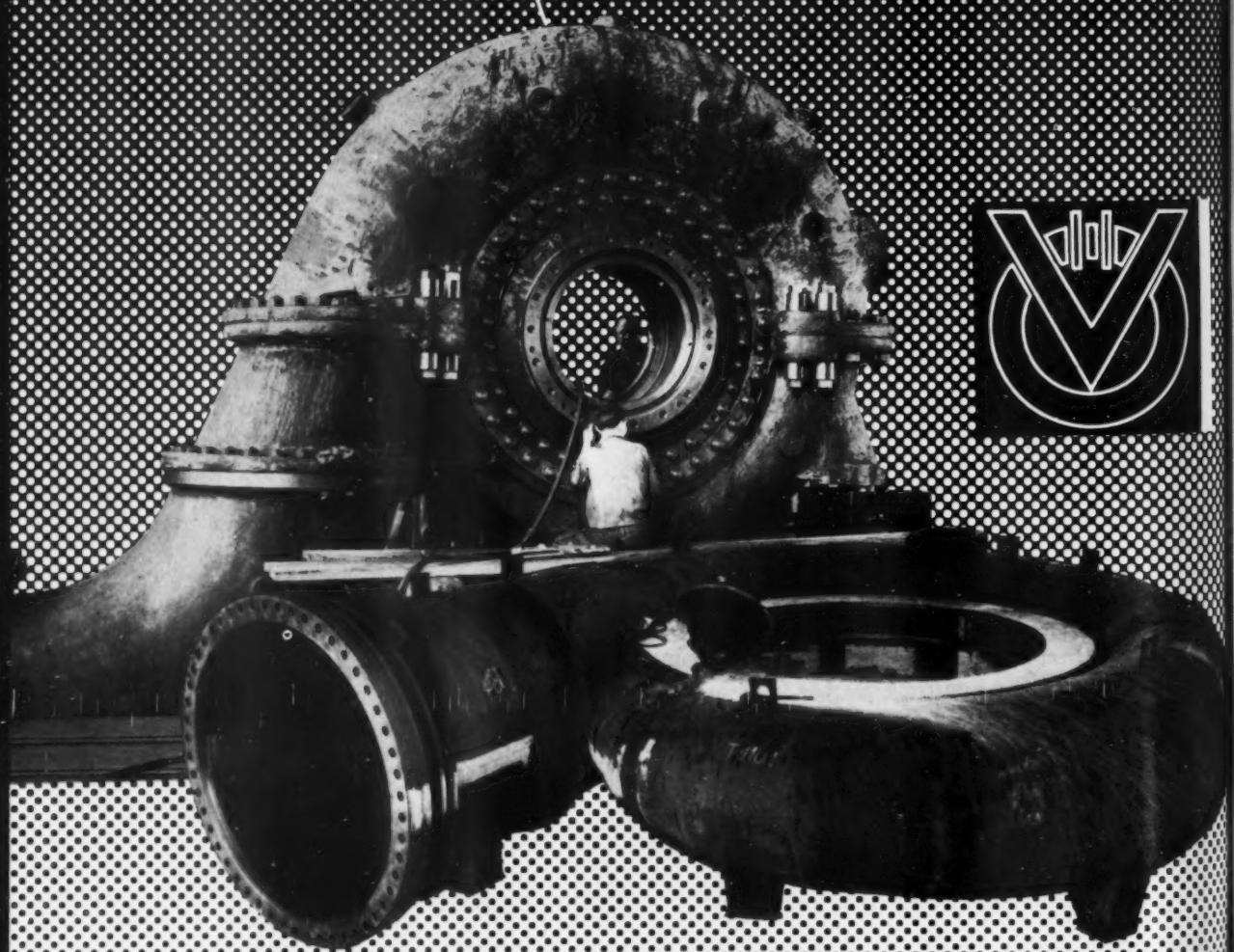

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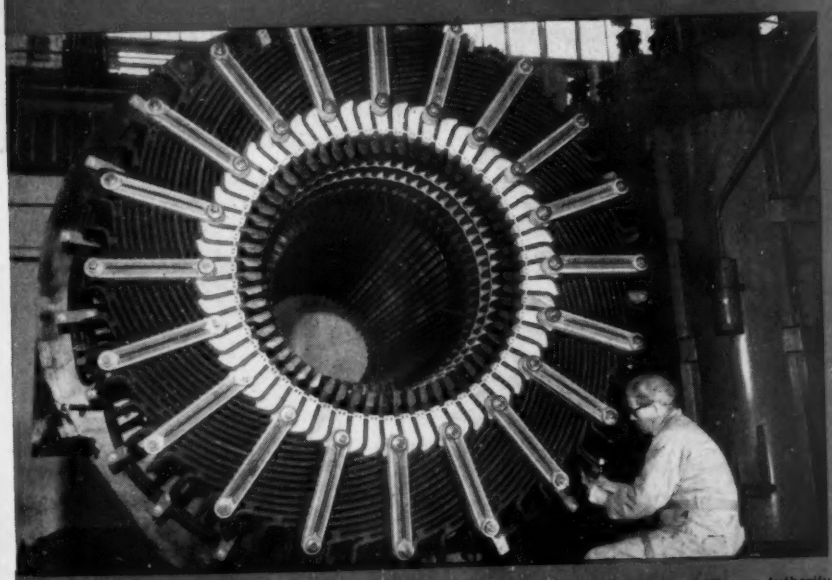

"PAXOLIN" and **MICANITE** are incorporated in the 23 MW air cooled turbo-generators at Chapel Cross Nuclear Power Station—another example of our service to the electrical industry.

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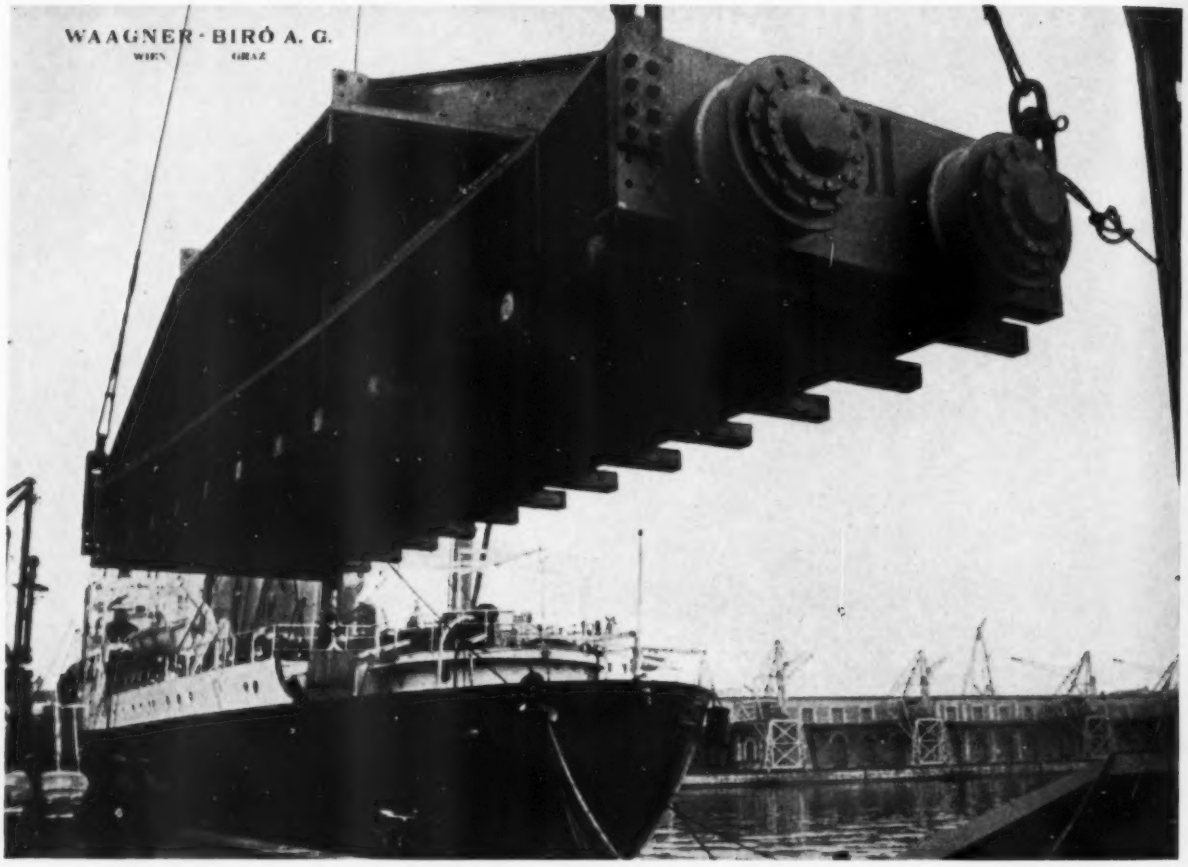

Transloading in the Port of Trieste of a part of an intake bulkhead gate for the Nile Power Station at Assuan. Five of these parts make up one complete bulkhead gate. Dimension of the gate 8 x 14 m.

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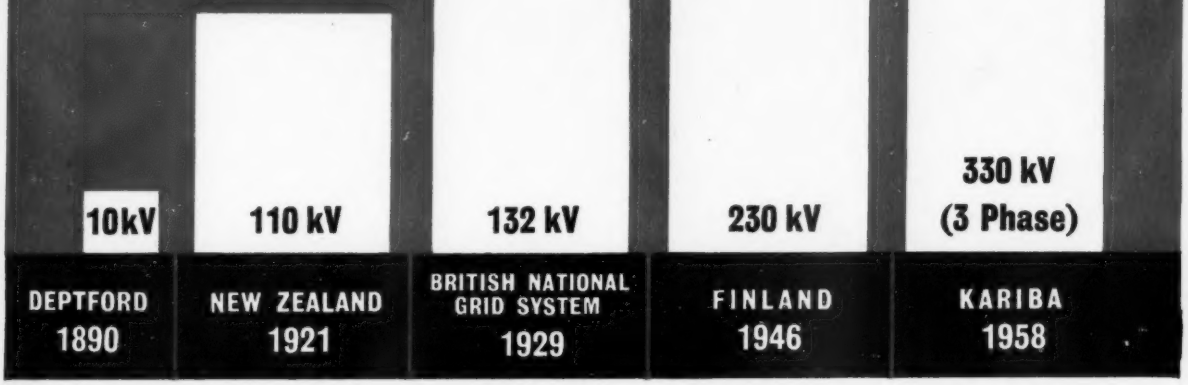

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The skill of Ferranti engineers over a period of more than 65 years has resulted in the award of contracts for the supply of transformers of the highest voltage and rating for the world's largest electric power projects.

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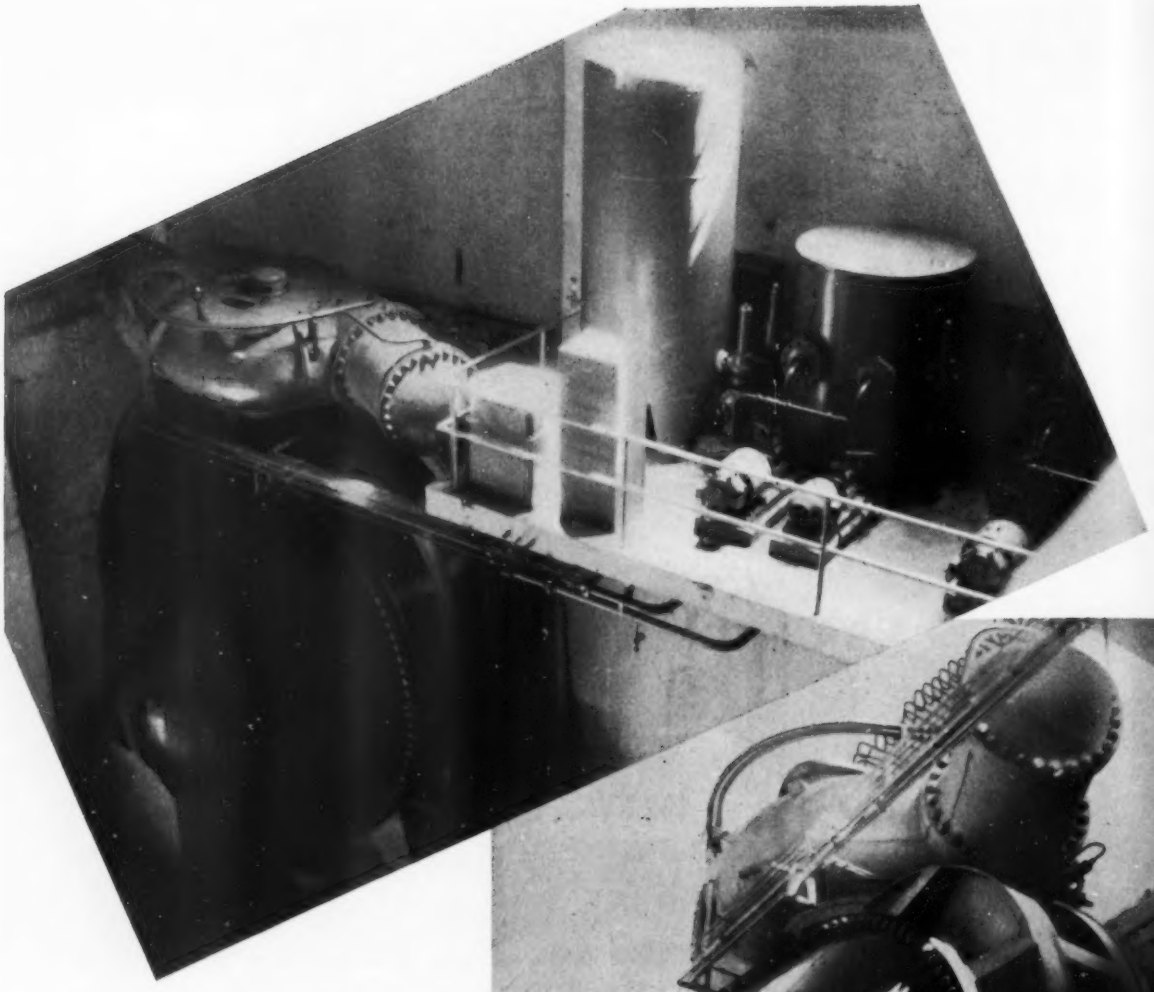

Butterfly valve with provisional downstream bulkhead

SEYHAN WATER PROJECT

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THREE BUTTERFLY VALVES

- Weight of valve leaf: 110,000 lbs.
- Diameter of valve leaf: 17 ft.
- Maximum Head: 150 ft.
- Delivery: 2,600 cu. ft./sec.

FRANCO TOSI S.P.A.
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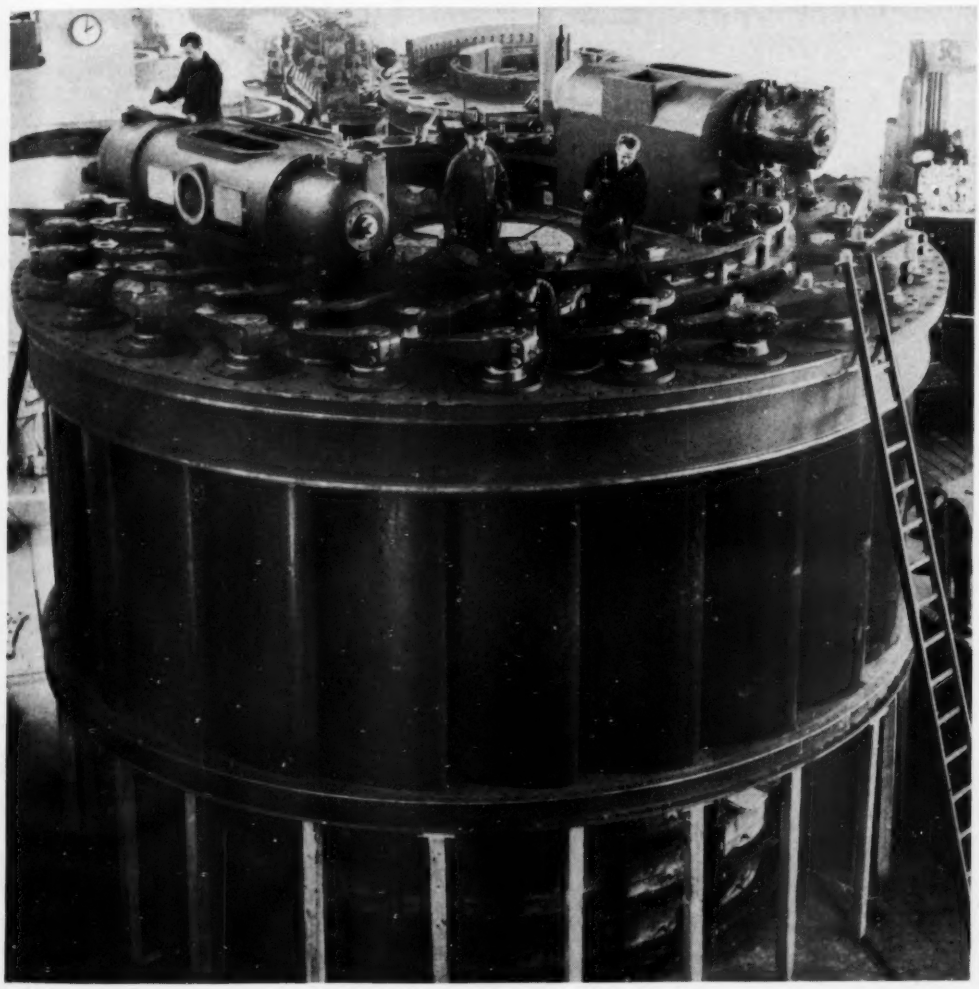
South America's largest Kaplan turbines to be delivered by NOHAB



Shown here is one of the gate operating mechanisms for the Rincón de Baygorria Power Station, at the NOHAB works, Trollhättan, Sweden.

The three Kaplan turbines for the Rincón de Baygorria Power Station, situated on the Rio Negro, Uruguay, are now under construction in our shops.

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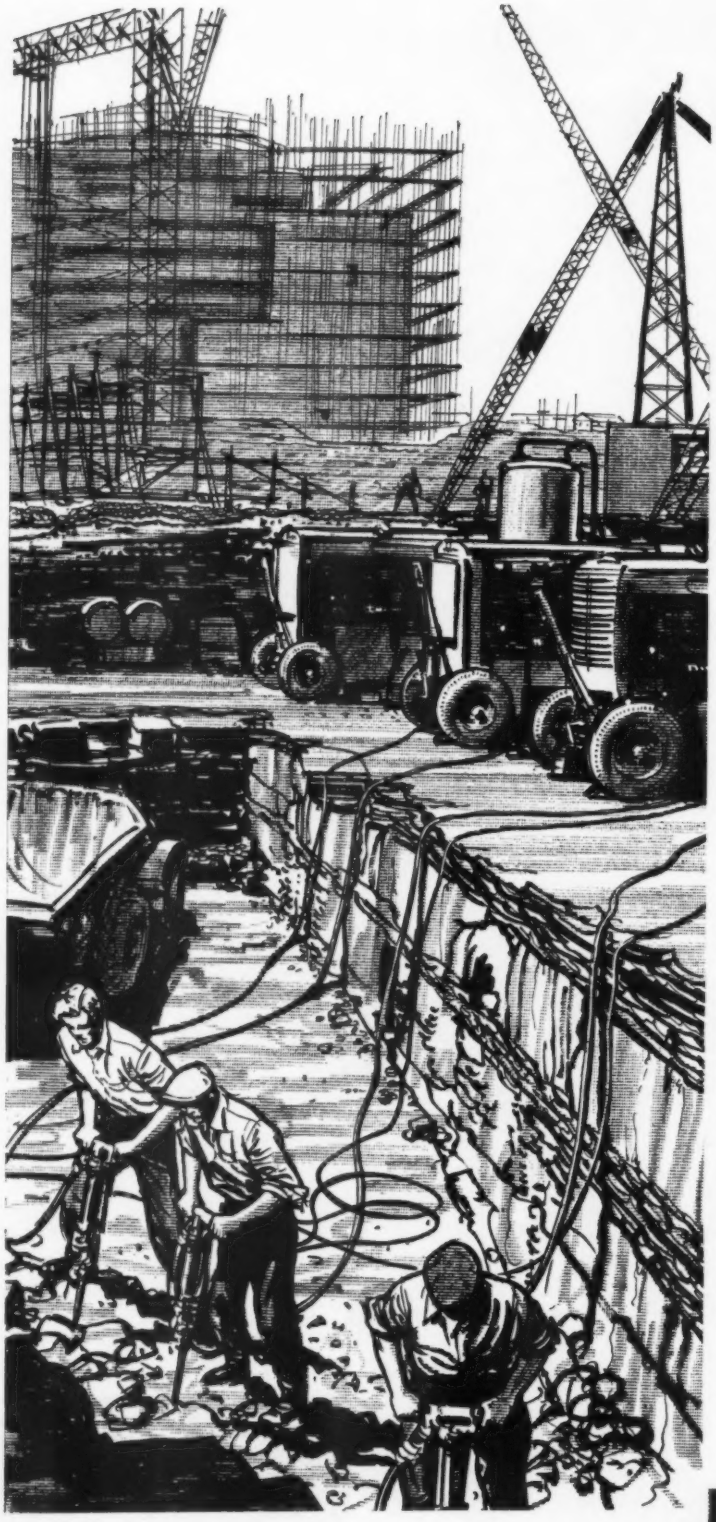

Taylor Woodrow chose Holman

Reliability pays at Hinkley Point

The world's largest atomic power station, with a capacity of half a million kilowatts, is taking shape at Hinkley Point, Somerset. The contract, for the Central Electricity Generating Board, is being carried out by the English Electric, Babcock & Wilcox and Taylor Woodrow Atomic Power Construction Co. Limited.

Compressed air equipment plays a very important part in its construction; Taylor Woodrow chose Holman for the job. Batteries of compressors, rock breakers and chipping hammers with the well-known Holman reliability built into the design are taking shift after shift of rigorous work without faltering, beating construction schedules and slashing operating costs.

In major projects all over the world Holman units are proving their reputation over and over again. For sheer reliability and low running cost, you can't beat Holman.

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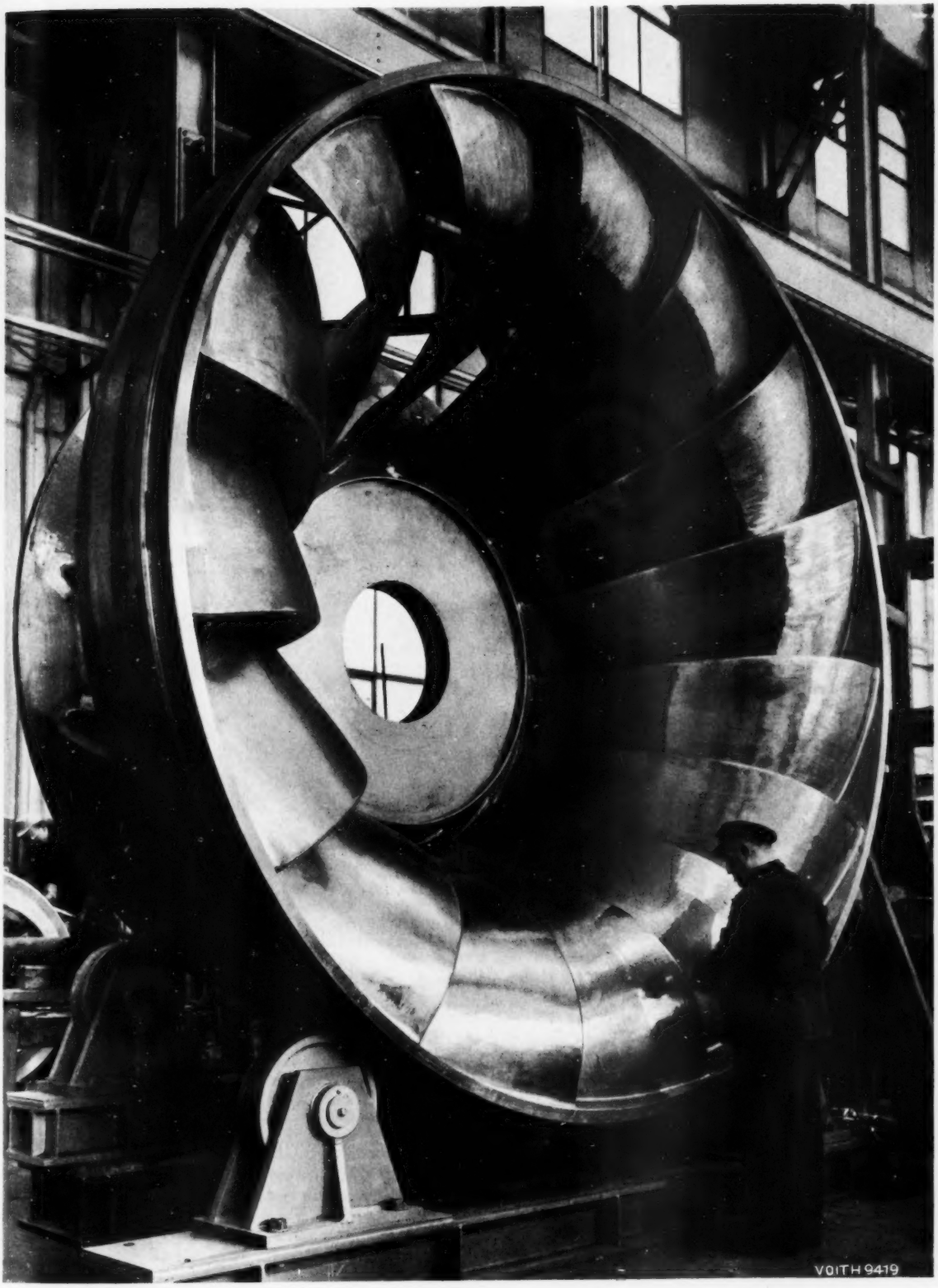
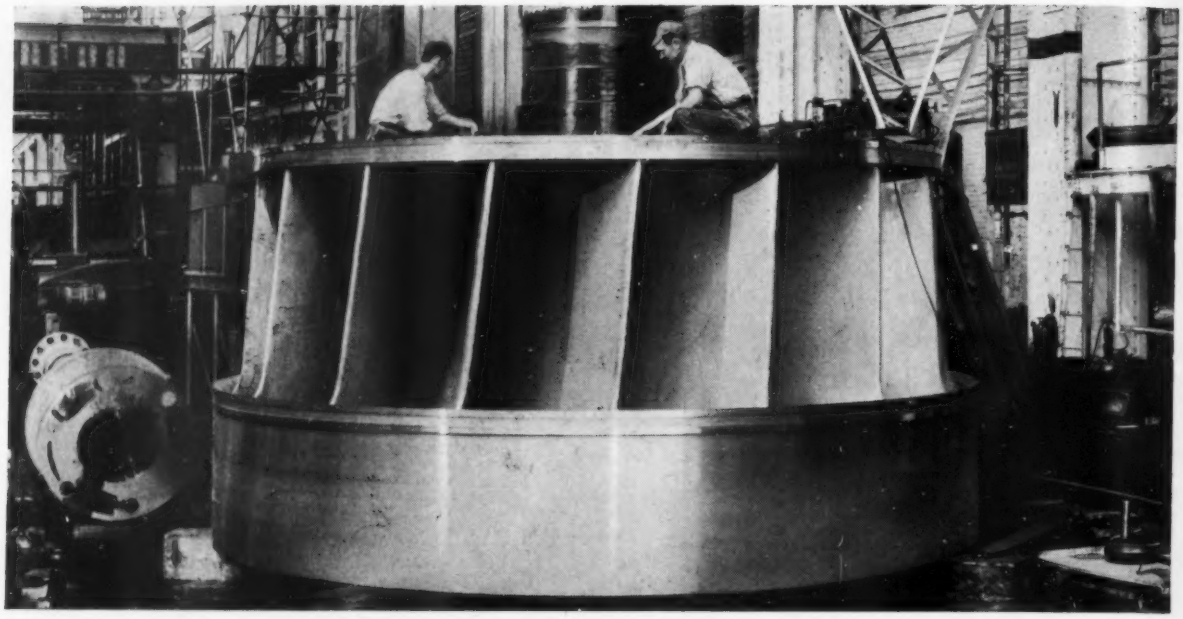
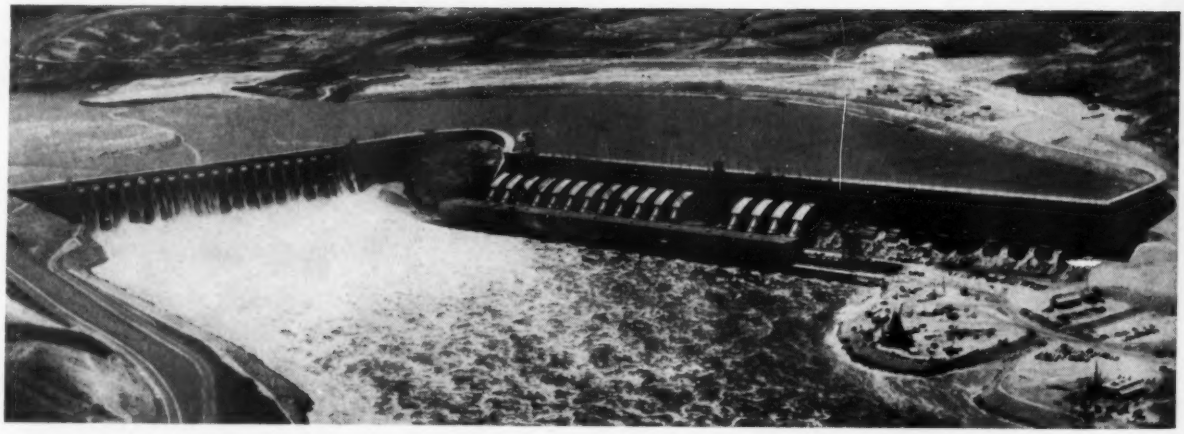


Voith Francis Runner developing 86.500 HP for Saucelle Power Station in Spain

J. M. VOITH GMBH · HEIDENHEIM (BRENZ) · GERMANY



Newport News builds 10 unique runners



For Chief Joseph Dam in Washington State

Cast buckets are welded to cast crown and band—then stress relieved

The Francis-type runner above illustrates Newport News' complete facilities for fabricating heavy equipment.

It also illustrates the specialized techniques Newport News frequently uses in jobs of this type. Gibson tests for these units are very favorable.

Before you go ahead on a project like this, be sure to get a bid from Newport News. Whether it's penstocks, butterfly valves, pumps or turbines, you'll get the benefit of the modern plant methods and production facilities of one of the country's oldest and largest builders of water power equipment.

Engineers... Desirable positions available at Newport News for Designers and Engineers in many categories. Address inquiries to Employment Manager.

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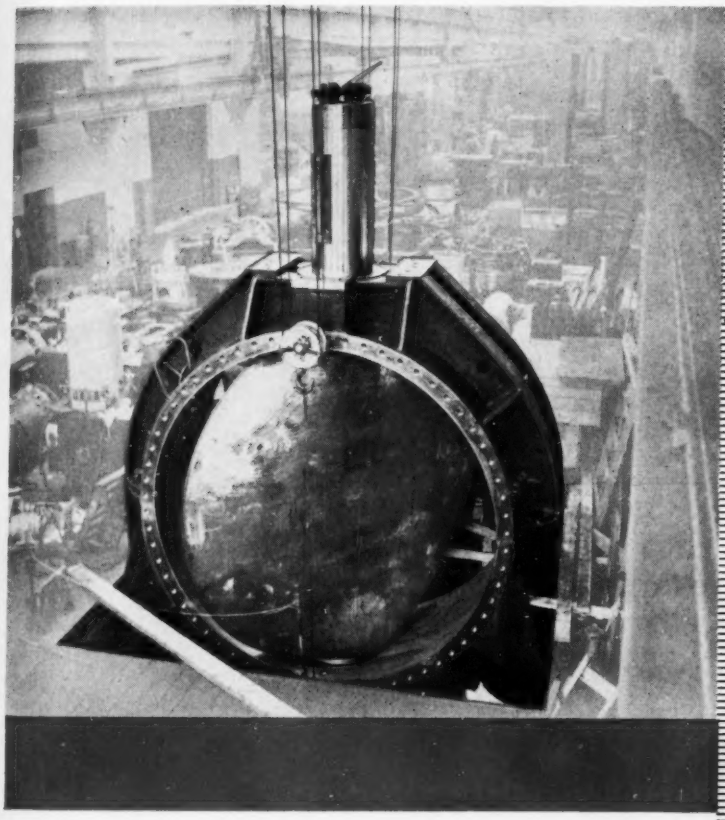
A Butterfly Valve

of remarkable size



The Butterfly Valve for one of the two KMW Francis turbines ordered by the well-known Swedish company, Stora Kopparbergs Bergslags AB, for their Trängslet Power Station is now under erection.

The complete valve weighs 90 tons and the valve blade alone 27 tons. The latter has a diameter of 4.5 m (14 ft. 9 in.). When the valve is shut, the water pressure on the blade will be 2,400 tons.

The first turbine is estimated to be put into service in October this year and the second unit about four months later. The total output of the two turbines will be 280,000 h.p. at 140 m (459 ft.) net head and 214 r.p.m.

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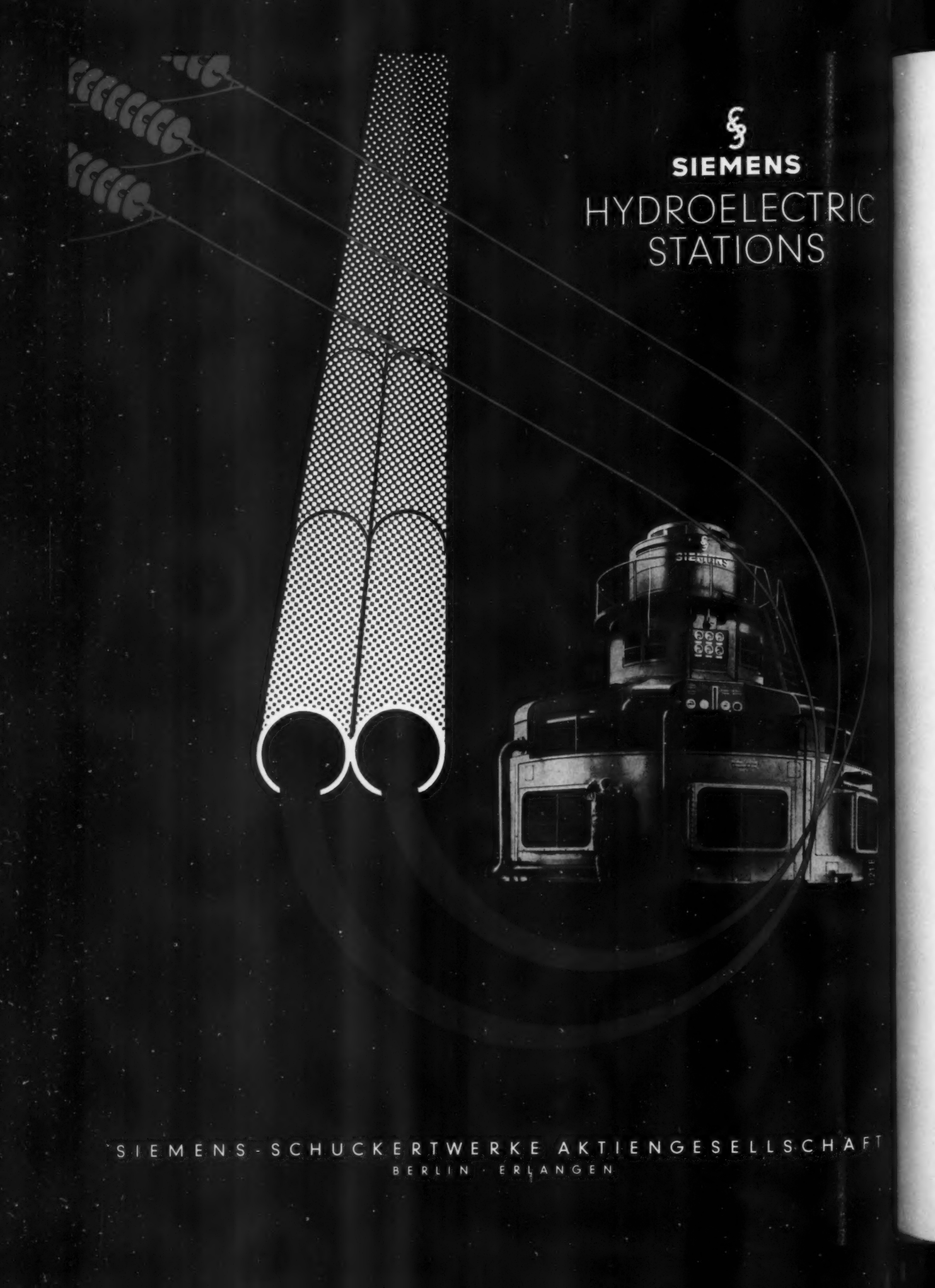

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Hydraulic Turbine Design

THERE is a popular misconception that the design of hydraulic turbines reached an ultimate stage of perfection many years ago and that any improvements in the comparatively recent past have borne only on matters of detail without materially altering efficiency. This is, of course, quite untrue as is evidenced by the great increase in efficiencies since, in 1907, Graf and Thoma plotted a curve to show the maximum efficiencies of a selected group of turbines installed in some European power stations.

The design of conventional types of hydraulic turbine has improved steadily over the past 50 years and the trend of research has been to develop faster, and thus smaller and cheaper, units conjunctly with higher efficiencies. As in many other branches of engineering, technical achievement in this field has to some extent advanced in parallel with metallurgical and machine-shop practice. For instance, the trend in the design of Pelton wheels is to use integrally cast runners rather than separate buckets and this has only been made possible by the great advance in steel foundry techniques during the past 30 years.

A very interesting appraisal of some aspects of the evolution of hydraulic turbines during the present century was the subject of a James Clayton lecture presented by M. Pierre Danel at a meeting of the Institution of Mechanical Engineers in London in December 1958. M. Danel, who is President of the Société Grenobloise d'Études et d'Applications Hydrauliques at Grenoble, is an eminent engineer and has been very closely associated with advanced research in hydraulics for over three decades. During the lecture M. Danel showed how specific speed can be expressed as $ns = n_{1-1}\sqrt{N_{1-1}}$ where n_{1-1} is r.p.m. of unit runner under unit head, and N_{1-1} is power in h.p. of unit runner under unit head. This expression cannot be further simplified and although the two terms in the expression are apparently independent, they are in fact closely related by a number of common conditions so that, from a practical viewpoint, an empirical term F can be used to express this relationship. The empirical function F, however, proves to be different according to the type of turbine, so that the specific speed is a good parameter to compare two turbines of similar design but may lead to a false impression when used to compare units of totally different types.

In recent years there has been a steady advance in the design of Pelton and Francis turbines, and the overlap between the fields of these two types has increased to such an extent that it is becoming more and more difficult to decide which, under certain conditions, should be installed for a given project. An instance of this problem has occurred in Scotland at Finlarig (see WATER POWER, November 1958) where, if the present stage of development in the design of Francis turbines had been reached when the power station was conceived, a Francis unit might well have been used in preference to the existing unit, a 30 MW Pelton turbo-generator operating at 375 r.p.m. under a net head of 1,275 ft.

Francis turbines were at one time used for specific speeds up to 450 metric, but now that the Kaplan has been developed and perfected, they are no longer built with specific speeds of more than 350. There is also an overlap between the Francis and the Kaplan type and, in recent years, the latter has been built for higher and higher heads with correspondingly lower specific speeds. Kaplan turbines are now used for heads between 30 and 50 m., a practice which would have been considered very difficult some 20 or 30 years ago.

A further evolution of the Kaplan-type runner has been the bulb unit, a subject which has been discussed on many occasions in this journal. The ideal turbine for the harnessing of tidal power should operate in each direction, both as a turbine and as a pump, and it is only because the bulb unit has been developed that projects for harnessing tidal power are now feasible.

Many statements have been made in the past to the effect that hydro-electric turbines have attained their maximum efficiency, but M. Danel has said that there

may well be much further progress in the design and operation of hydraulic turbines in the future.

The James Clayton lecture is presented once or twice every year by an engineer or scientist who has merited the highest honours in the field in which he has specialised. These lectures, sponsored by a trust established by an eminent British engineer, James Clayton, are intended to stimulate interest and to instruct the individual so that the advance in techniques and the progress in specific fields may be more widely propagated throughout the engineering profession. M. Danel has most ably fulfilled this purpose and, in so doing, he has received some recognition of the great contribution that he has made to hydraulic research.

Fortun Hydro-Electric Scheme

THE Fortun scheme involves the construction of 14 dams, 12 subsidiary tunnels and one main tunnel, which is 12·5 km. long, a headrace 12 km. long, two penstock tunnels each 1,500 m. long, and a tailrace tunnel 500 m. long. The Fortun river flows in a southerly direction and discharges into the Sogn fiord, the longest fiord in Norway. This river system has a catchment area of 280 sq. km. and when the power scheme is fully developed a storage capacity of 245 million cu. m. will be provided. At the site chosen for the main power station a head of 1,000 m. will be available and a regulated water flow of 22 cu. m. per sec. There will be three stages and the completion of the first plant (Fortun I) is expected to be ready in May of this year and will have a capacity of 150,000 h.p. Fortun II station, which will be of the same capacity, will not be ready until the summer of 1961. Designs for the Fortun III station have not yet been finalised but will probably involve pumped storage. The scheme is being financed by A/S Ardal og Sundal Verk, the principal aluminium producer of Norway, and constructional work was begun in the autumn of 1956.

Pozzolana for Dams

THE New Zealand Department of Scientific and Industrial Research has investigated the pozzolana found in New Zealand and this research has culminated in the award of a contract for grinding local diatomite, by micronising, preparatory to its use in the Atiamuri and Ohakuri hydro-electric projects on the Waikato river, North Island.

Pozzolana is a finely divided siliceous material, which, though not cementitious in itself, will combine with lime at ordinary temperatures and, in the presence of water, form stable insoluble compounds possessing cementing properties.

An evaluation of natural materials occurring in New Zealand to determine their pozzolanic properties was begun in 1951. Of a wide range of materials tested it was found the two most promising types of pozzolana in the country are diatomite and pumicite. Two diatomites and two pumicites were selected for pilot-scale tests in actual concrete mixes and, of these materials, three were from the Waikato area, the fourth from North Otago. More comprehensive tests were then carried out and it was found that diatomite is more reactive than pumicite. Numerous physical measurements gave information regarding the effect of the materials on the properties of plastic and hardened concrete, and showed the optimum percentage replacements of the pozzolana, the strengthening effect of the fine grinding of pumicite, and also indicated that when used in air-entrained mixes, the drying shrinkage of the concrete can be controlled satisfactorily. The reduction in early heat generation in concrete was also demonstrated.

Although the use of pozzolana dates back to the days of the early Romans, with Portland cement available as a major constructional material, they have, until the last few years, found only limited use. The modern practice is to replace 15–35 per cent. of the cement in concrete by pozzolana. This may be done at the mixer or, alternatively, the pozzolana may be blended with Portland cement after its manufacture.

Pozzolana is still being used today with lime alone; for instance, pozzolana lime in the ratio of $3\frac{1}{2} : 1$ was issued in the construction of the large dry dock at Naples.

Pozzolana concretes have greater resistance to seawater corrosion than normal concretes, and in Italy, where pozzolana is freely available, its use in the building of marine structures is compulsory. It is, however, in hydro-electric projects that it is of the greatest advantage; one reason for this is that the heat generated in the dam is greatly reduced when pozzolana replaces part of the concrete.

Company to Control Dominican Projects

A PROPOSAL for the creation of a public or private organisation to take charge of the financing, operating and administration of the £$5\frac{1}{2}$ million Tavera dam and similar hydro-electric projects in the Dominican Republic has been made in a recent symposium on the natural resources of the Republic held in Cidad Trujillo. It was stated by Dr. Fidel Nunez that great progress had been made on research and codification of information regarding possible hydro-electric and irrigation schemes. A single authority to take charge of the Tavera dam and further dams would enable economies to be made and give a unified approach to these developments.

I.A.H.R. 1959 Congress

THE Eighth Congress of the International Association for Hydraulic Research will be held at the Windsor Hotel in Montreal, Canada, during the week beginning August 24, 1959. The following four subjects were selected for study in the technical sessions by the General Assembly at the Lisbon Convention in 1957: (a) Hydraulics of Gates and Valves (including vibration, forces, cavitation, ice); (b) Fundamental Hydraulics of Ship Locks; (c) Density Currents (including effects on reservoir sedimentation, hydro-electric intakes, fishing industry, etc.); (d) Air Entrainment and Air Vents. Two seminars for more informal study will be conducted concurrently with the technical sessions on the following subjects: (1) Ice Problems in Hydraulic Structures; (2) Transportation of Material in Water. (It is intended that this seminar shall deal primarily with sediments but transport of other materials may be included.)

In addition to the technical sessions, excursions are being planned to the St. Lawrence Seaway structures,

to some of the large hydro-electric projects of the region and to hydraulic laboratories that have contributed to the development of both. In this connection it is to be noted that the St. Lawrence Seaway Project will be officially opened in 1959.

All communications regarding attendance, participation, submission of papers, etc., should be addressed to Mr. Leo Roy, c/o Quebec Hydro-Electric Commission, 107 Craig Street West, Montreal 1, P.Q., Canada.

Noxon Rapids Dam Finished

NOXON Rapids dam, built by Morrison-Knudsen Company, Inc., reached crest level in November 1958, no less than 15 months ahead of schedule. It has been erected on the Clark Fork river, Noxon, Montana, and the associated power house will eventually have a capacity of 400 MW. The contract for the dam was awarded to the M-K concern by The Washington Water Power Company and has a crest length of rather more than one mile and a maximum height of 265 ft. The spillway and power-house sections have a combined length of 1,130 ft. and contain some 560,000 cu. yards of reinforced concrete, while the adjoining embankments contain about 3 million cu. yards of earthfill. The power-house structure, which is of the open type, is 500 ft. long, 120 ft. wide, and will house four 100 MW generating units with space for a fifth.

The Peace River Project

ALTHOUGH this project is still in a very preliminary stage, a new company known as the Peace River Power Development Co. Ltd. has recently been formed to continue investigating the hydro-electric potential of the Peace. The preliminary engineering surveys have been promising and they indicate a potential of not less than four million h.p. on the Peace river itself above Hudson Hope at a low cost per kWh at site and at a below-average cost in the lower mainland area of British Columbia. A year or more of detailed engineering remains to be done before a firm decision can be made and a final plan for development of the Peace river can be formulated.

There are only three rivers that can be further developed for power production in British Columbia—the Fraser, the Columbia and the Peace. The Fraser would provide the cheapest power but no solution has yet been found to the salmon problem although research during the past three years has produced promising results and it may well be possible ultimately to harness the power potential of this river.

The situation on the upper Columbia is complicated by the need for full international agreement on any hydro-electric project for development of this river and, in the past, it has proved difficult to reconcile satisfactorily the interests of the two countries concerned. There has, however, been some progress in this respect very recently and although power from the Columbia must necessarily be problematical for the time being, this river will certainly be developed eventually.

If the present investigation of the Peace river is favourable, the first power should be available in the mid-1960's. The present shareholders in the Peace River Power Development Co. Ltd. are the Western Development and Power Ltd. (a wholly-owned subsidiary of the British Columbia Power Corporation Ltd.), Wenner-Gren Development, Power Securities Corporation Ltd., the National Bank City Office Nominees Ltd., and Associated Electrical Industries Ltd.

New Swedish Plants

AFTER appropriate test runs, the Bålforsens Kraft AB have now accepted the two KMW Kaplan turbines for their Bålforsen power station. Despite difficulties of various kinds it has been possible to complete the power plant according to schedule. The turbines have a combined output of 102,220 h.p. at a head of 31 m. (102 ft.) and a speed of 150 r.p.m. This power plant on the Ume River in northern Sweden is housed in an underground station.

The third KMW turbine has been added to the Krångede AB's power station at Ramsele and is now in full-time operation. The first two Francis turbines of this plant were put into service last summer and the third turbine completes this project which is now capable of producing 206,490 h.p. at a head of 74 m. (243 ft.) and a generator speed of 187 r.p.m. The runners are integrally cast in stainless steel.

Two of the four double horizontal propeller turbines ordered for Lottefors power station have now been put into service. They are designed to supply power to a woodpulp grinding mill owned by Marma-Långrörs AB at Lottefors, near Söderhamn, a coastal town in central Sweden.

Progress at Hirfanli Dam

HIRFANLI dam, the largest and most important of the seven dams in the Kizilirmak project in Turkey (see WATER POWER, January 1957), was completed ahead of schedule on September 13, 1958. The height of the dam from river bed to crest is 81 m., the length at the crest is 364 m., and the total volume, including rock, impervious core, and semi-pervious material, amounts to over two million cu. m.

The first 32,000 kW vertical-shaft turbine has also been completed but the reservoir level is still 15 m. below the minimum head required, although it is expected that adequate water will be available by the spring of 1959. The switchyard and intake structures have also been finished and the spillway weir and bridge are about 70% complete. Work on the tunnels is now in the final stages and the last penstock sections are being welded. Structural work in the power station was finished over two months ago and it is hoped that the second and third turbines will be installed early in 1959.

The Hirfanli project is a joint venture by the English Electric Co. Ltd. and George Wimpey & Co. Ltd. Tippetts-Abbett-McCarthy-Stratton, of New York, are the consultants.

Rosport Power Station, Luxembourg. In our October, 1958, issue we published an Editorial Note about the Rosport and Esch sur Sûre power stations in Luxembourg. We regret that we were mistaken in our reference to the electrical equipment for Rosport power station. All the electrical equipment for this station is being supplied by Siemens-Schuckertwerke A.G., Erlangen.

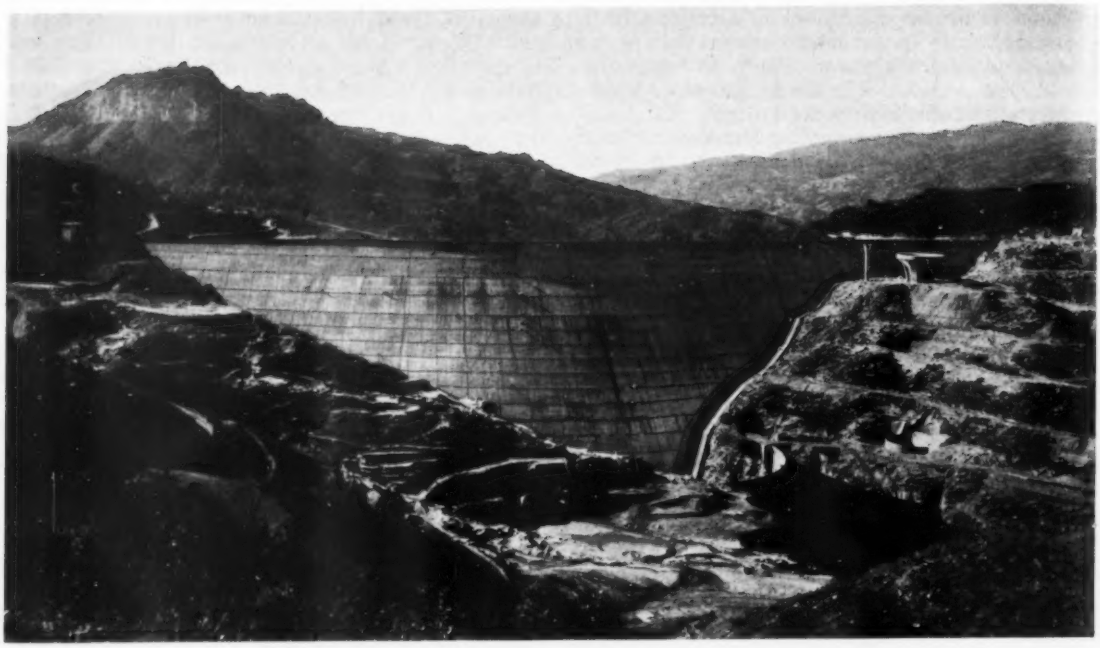


Fig. 14. Paradela dam seen from upstream, with the reservoir emptied for inspection after the first filling. The quarry can be seen on the mountainside in the distance

Recent Construction on the Cávado

In this concluding article we describe Paradela dam, claimed to be the highest rockfill structure in the world, the spillways, supply tunnel and associated works, and the extension to Vila Nova station, where interesting constructional problems had to be solved

PART TWO

WE commenced our previous article, it will be remembered, by outlining the various stages in the development of the Cávado basin. Chronologically, the first stage was the building of Vila Nova station supplied from Venda Nova reservoir on the Rabagão. Secondly, Salamonde station was built downstream of Vila Nova, and thirdly, Caniçada station, described last month, was built downstream of Salamonde. The fourth and fifth stages have been designed to complete the development of the upper part of the catchment. Stage four consists of creating a large reservoir on the Cávado at Paradela to feed an additional set in Vila Nova station, and stage five, which will be the final one, aims to create a pluriennial storage on the headwaters of the Cávado and Rabagão, together with a further power station which will include pumped storage.

It is our purpose in this article to describe stage four, which has just been completed; for stage five the works for the temporary diversion of the river are being done and the contractor's proposals are being studied. Stage four presents many points of technical interest. Paradela dam is believed to be the highest rockfill dam in the world and contains some unusual design features. It was constructed in three stages (actually a fourth stage was used but it was just a topping one, 3·5 m. high), and the spillway arrangements were designed to fit in with these three stages. The supply tunnel includes a section of steel pipe carried in a stretch across a valley on trestles. and the installation of the new generating set in Vila Nova station raised some interesting constructional problems.

Paradela Reservoir and Dam Site

Paradela reservoir, which covers the confluence of the Pitões with the Cávado, will provide a useful storage of 159·4 million cu. m. at a maximum retention level of 740 m., and drains an area of 126 sq. km. In addition to the main dam at Paradela, two side embankments were required to seal off the flooded area, at Telheira and at Barreiros, the former being 14·4 m. high and 162 m. long and the latter 7·9 m. high by 151 m. long. Both are of rockfill construction

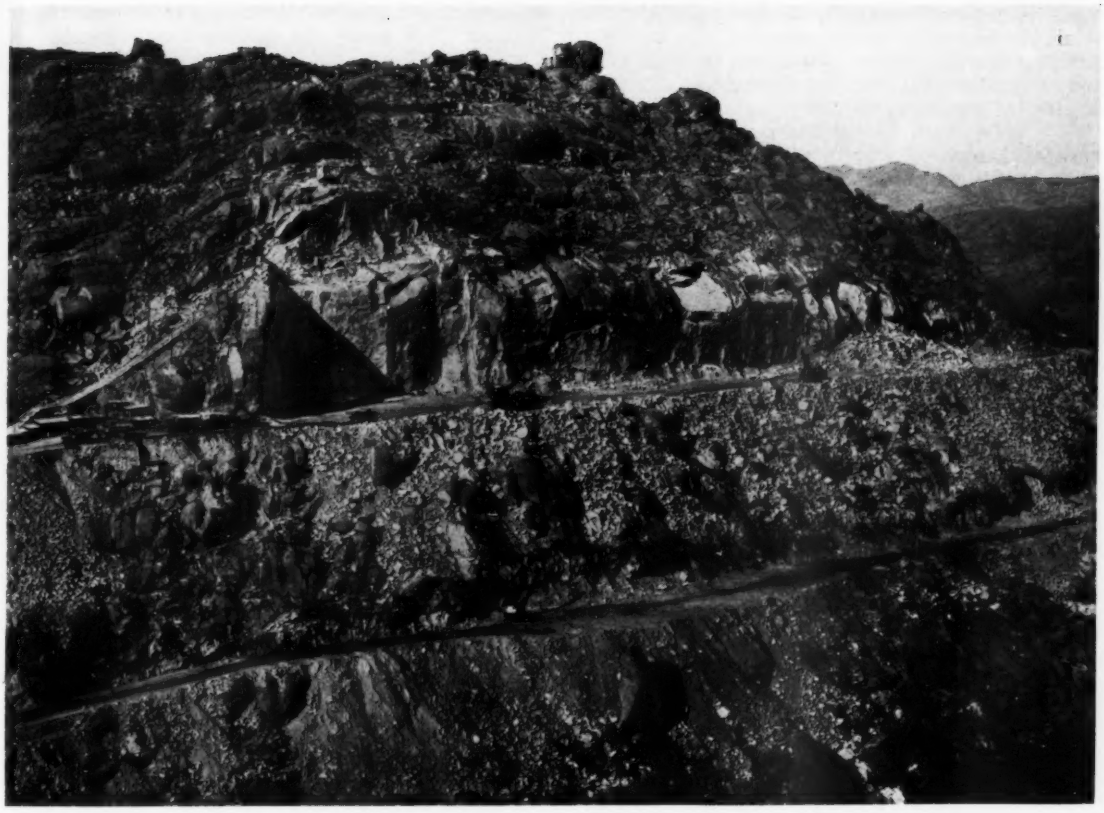


Fig. 15. Paradela quarry soon after the commencement of operations

with impervious concrete membranes on the water face, similar in principle to Paradela dam to be described in a moment. Telheira embankment is depicted under construction in Fig. 20.

Site investigations at Paradela soon disclosed that geological conditions were unsuitable for any form of concrete structure. The terrain consists of a very weathered granite, highly disturbed and fractured, which could not have been trusted to sustain the loads imposed by a concrete dam of either the gravity, buttress, or arch type. An earth dam was also ruled out because no suitable material was available in the vicinity, and the choice, almost of necessity, fell on a rockfill structure. Fortunately, a site at which an ample supply of excellent granite was to be found was proved only about 2 km. away from the site of the dam, and a vital part of the construction programme was to open up a quarry on this site capable of the required output of rockfill, and the construction of a road adequate to sustain the intensive lorry traffic between the quarry and the dam site. In all, Paradela dam required the removal of nearly two million cu. m. of solid rock, the construction programme on the dam being about 120,000 cu. m. of sluiced fill and 10,000 cu. m. of placed rock each month.

The Quarry

A general view of the quarry appears in Fig. 15. The quarry was laid out in two benches, with a maximum aggregate height of 100 m., the lower being 40 m. high and the higher up to 60 m. Shot-holes, 10 cm. in diameter, were sunk by a novel form of pneumatic drill developed by Stenuick Frères, Fontaine-l'Eveque, Belgium. In this design the drill body, containing the hammer and valve and carrying the cutting head, forms a self-contained unit which is lowered bodily down the hole, the rods forming the drilling string being hollow and conveying the air supply to the drill. The exhaust air is used to blow the cuttings out of the hole. Rotation is effected by an air motor on the drilling rig. With this arrangement the hammer impacts are not transmitted through the drilling string—which is thereby relieved of the corresponding stresses—but are applied directly to the cutting head, so that drilling is equally effective at any depth. This is reflected in the low air consumption, which is less than half that of an equivalent wagon drill.

The quarried rock was loaded out by five shovels, two of 4 cu. yd. bucket capacity and three of 3½ cu. yd., one shovel being kept in reserve. All five shovels were of Lima manufacture.

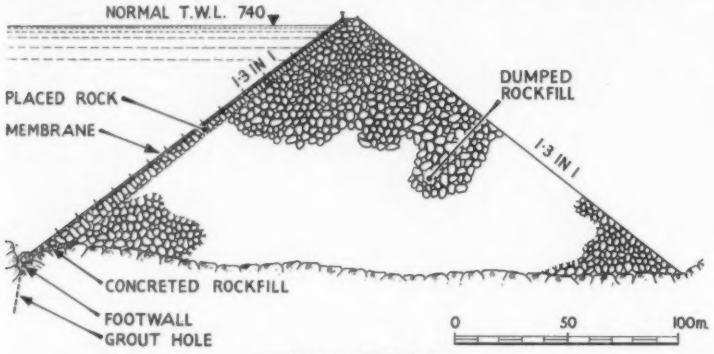
To transport the rock from the quarry to the dam a fleet of 22 Blackwood-Hodge 20 ton Euclids was employed. To enable loads to be controlled safely downhill, these vehicles were equipped with supplementary brakes consisting of an oil pump discharging through a controllable orifice, known by "hydrotarders."

The road between the quarry and the dam site was specially built to cope with the heavy traffic involved, aggregating at peak about 500 Euclids in 24 hours. It is 4½ km. long, 9 m. wide, and is asphalted. Cross

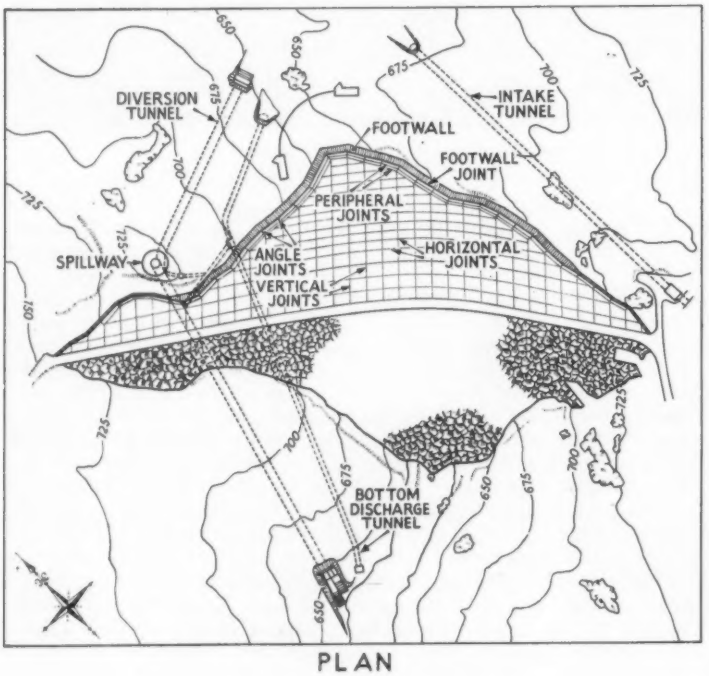
roads have been avoided by the use of bridges. To obtain the requisite output, quarrying, transport, and dumping of rockfill, proceeded in two ten-hour shifts, and to facilitate this the quarry, road and dam were floodlit by sodium lights.

Paradela Dam

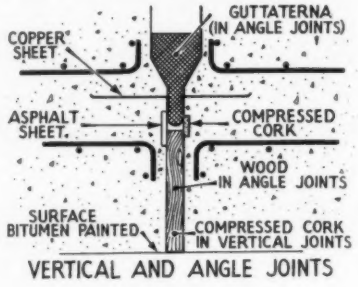
To enable the new generating set in Vila Nova station to produce power as soon as possible, Paradela dam was built in three stages. The first, which was reached in October, 1956, provided pondage at a level of 672·5 m., enabling the generating set to operate on a run-of-river basis; the second stage, reached in October, 1957, raised the maximum retention level to 730 m., and the third stage, completed in April, 1958, afforded storage up to the final normal elevation of 740 m. The rockfill construction, incorporating the use of an impervious membrane on the upstream face, enabled these three stages to succeed one another without encountering any of the difficulties that arise when heightening a concrete dam. As completed, the dam is 110 m. high and has a crest length of 532 m., thus



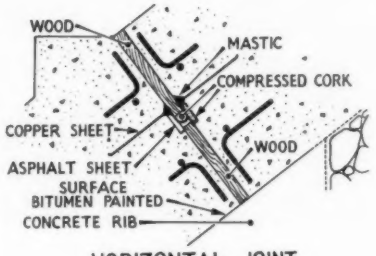
TYPICAL SECTION



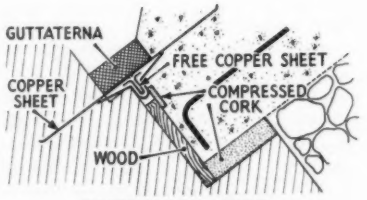
PLAN



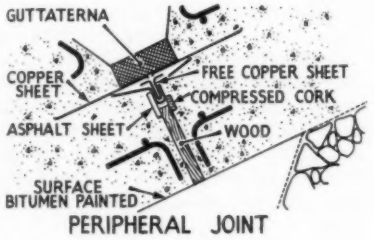
VERTICAL AND ANGLE JOINTS



HORIZONTAL JOINT



FOOTWALL JOINT



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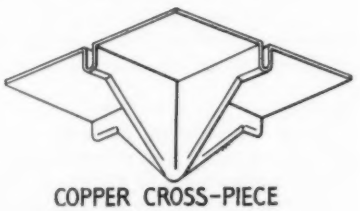
COPPER CROSS-PIECE

Fig. 16. Plan and typical section of Paradela dam, with details of the joints used in the construction of the impervious membrane

Fig. 17. Stone packing being placed in readiness for the concrete membrane

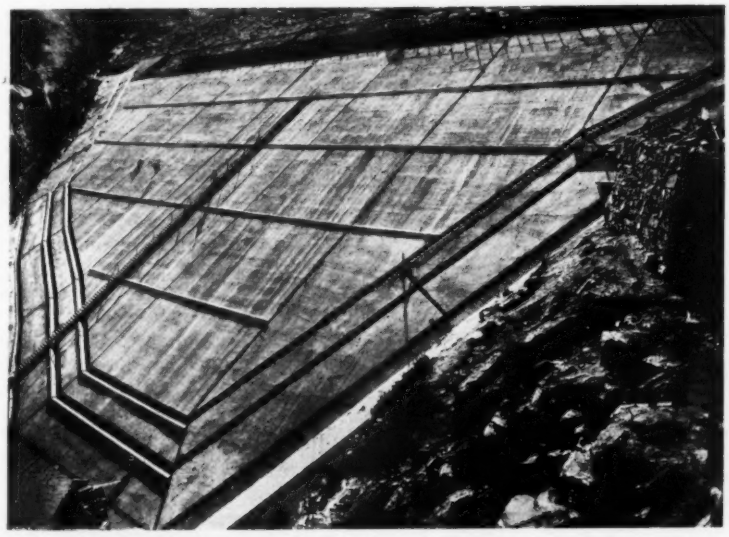

Fig. 18. Lowest portion of the concrete membrane completed

exceeding by 10 m. the height of the dam at Salt Springs, California, which was previously the highest dam of this type in the world.

The design of Paradela dam was, in fact, inspired by the successful experience at Salt Springs, and a team of HICA engineers was given facilities to visit several rockfill dams in the U.S.A. and study their designs before the plans for Paradela were finalised. A number of original features are to be found in the design of Paradela dam, but the opportunity to examine those dams naturally proved of the utmost value.

A plan and section of Paradela dam are given in Fig. 16. An inclination of 1·3 : 1 was chosen for the upstream face and the downstream face was allowed to build up naturally to the natural angle of repose of the rock. The upstream toe of the dam consists of a concrete footwall containing an inspection gallery from which a grout curtain was put down, long inclined holes being sunk to a maximum length of about 25 m. and short inclined holes, both upstream and downstream, to a length of about 5 m. and 2 m., establishing two auxiliary curtains to reduce the leakage of the grout of the main curtain.

This footwall also acts as a sill for the impervious membrane constituting the water face of the dam. Below the membrane itself is a layer of placed rock ranging in thickness from 3 m. at the top to 8·6 m. at the bottom, both of these measurements being taken normal to the face; the horizontal thickness at the bottom is about 14 m. This layer was built with the largest blocks of granite —ranging up to 10 tons each— that could be transported from the quarry, and so placed as to afford the maximum contacting surface between blocks. They were placed by crane, and the larger voids between them were packed with smaller blocks not less than 20 kg. in weight. The remaining voids were filled with hard crushed rock about 5 cm. in diameter to form a securely packed structure.

The impervious membrane consists of reinforced-concrete panels about 16 m. wide and ranging from about 10 to 15 m. on the slope. The success or failure of the dam naturally depends on the behaviour of this membrane, both under the influence of the water pressure and under the movements caused by the natural settlement of the rockfill. These questions were studied in great detail in the light of the experience gained at Salt Springs, Bear River, and other American rockfill dams, and some interesting conclusions were drawn which were applied to the design of Paradela dam.

In the first place, a distinction must be drawn between the effects on the membrane of settlement in the horizontal and in the vertical directions. In a horizontal plane the sides of the dam must be regarded as rigidly fixed against the valley walls, so that if the dam face were made flat on a horizontal section any settlement would cause the joints between the concrete panels to open. To prevent this the face must obviously be made convex horizontally, but at Paradela, instead of adopting the conventional circular profile, the horizontal sections have been made hyperbolic. This has the effect of concentrating the curvature towards the centre of the dam, where the settlement is likely to be greatest.

Fig. 19. Paradela dam in the summer of 1957, showing the first stage flooded, the concrete membrane for the second stage being constructed, and the rockfill for the final stage being dumped

At first sight it might appear that a similar convexity should be adopted for the membrane in a vertical plane, but conditions can be shown to be entirely different, for the crest of the dam is not fixed but subsides as the rockfill settles. Thus the membrane has to be designed to withstand movements tending to build up compressive stresses in a vertical plane. To introduce a vertical upstream convexity, therefore, would result in the membrane attempting to part from its rock mattress and "swell up." This would be counteracted by the water pressure insofar as the reservoir was filled, and the resulting vertical compression might well be sufficient to crush the horizontal edges of the panels. At Salt Springs this has been guarded against by introducing a slight concavity in a vertical direction, but at Paradela the vertical sections of the membrane have been made flat, using special joints which will be described shortly, the water pressure being relied on to initiate some concavity as permitted by the settlement. Furthermore, the dam crest is made 2 m. higher at the centre than at the sides, to allow for settlement.

A particularly delicate problem arises at the junction between the bottom of the membrane and the concrete toe of the dam. This toe must be regarded as a completely rigid structure relatively to the face, so that the joint between it and the membrane behaves as a kind of hinge. At Salt Springs this had been found to result in cracking along this joint, and at Bear River a peripheral joint was introduced which greatly reduced the trouble. At Paradela, two peripheral joints have been used, one 4·5 m. away from the junction between the membrane and the toe and extending to an elevation of 725 m., and the other 3 m. away from the first and terminating at an elevation of 695 m.

A further phenomenon to be taken into account is a tendency for the rockfill at the sides of the valley to slide down towards the bed of the river. This action can be particularly troublesome in the case of the peripheral joints and the joint between the membrane and the toe of the dam, for it induces a shearing movement along the line of the joint. Details of the various joints are given in Fig. 16. Wood is used as the compression member, and is relied upon to squeeze to half its initial thickness without failure. Sealing is primarily by copper sheet, the edges of which are bedded into the concrete and the centre folded into the shape of a groove to accommodate movement. In cases where there is thought to be a possibility of this sheet being ruptured by shearing movement, it is pro-

Fig. 20. Telheira embankment under construction

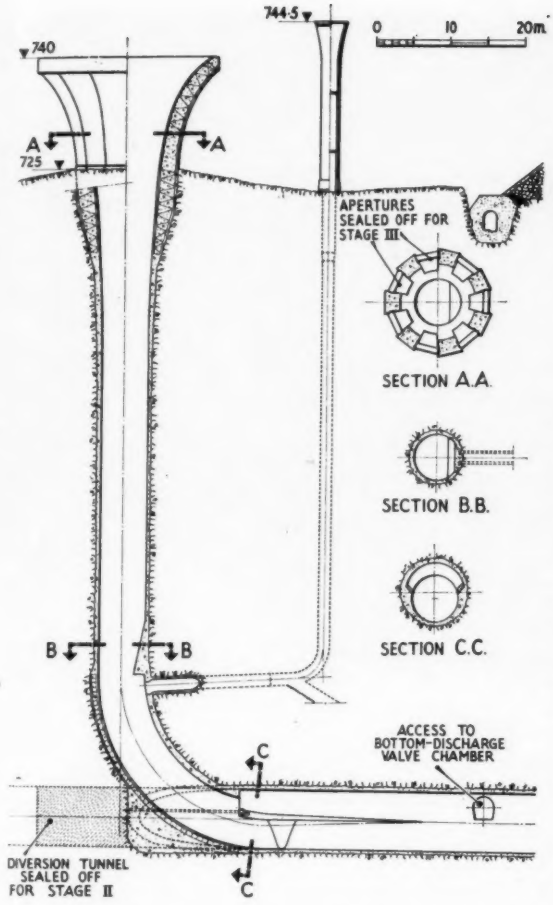

Fig. 21. Section through the main spillway, showing the three construction stages

tected by a block of unvulcanised rubber ("Guttaterna"), which adheres strongly to the concrete and to the copper sheet and can accept considerable movement. In these joints the copper sheet is also backed by a second copper sheet which is left free so that it is not distorted by sliding of the joint and avoids the "Guttaterna" being pressed in by hydrostatic pressure. To protect the "Guttaterna" from deterioration by the sun when exposed above the water line, wooden flaps are lightly secured over the joints. Finally, the junctions between horizontal and vertical joints are secured by special cross-shaped copper-sheet pressings of the form illustrated in Fig. 16.

Concrete was mixed in a Winget batching plant and placed by a 6 ton double cableway supplied by Etcheverry, Paris. A total of nine cranes was allocated to rock placing.

Spillways

The main spillway is of the uncontrolled morning-glory type. Its sill is at 740 m., and at the maximum expected level of 741·6 m. it discharges 304 cu. m. per sec. Previously, however, a model of this spillway was tested at the National Civil-Engineering Laboratory, Lisbon, and successfully dealt with a discharge of 850 cu. m. per sec. at the level of 743·10 m. Nevertheless, it was considered that as an absolute safeguard against overtopping so important a dam an emergency spillway should be provided, so a controlled spillway has been constructed capable of a discharge of 416 cu. m. per sec., so that a total flood with a peak of 1,550 cu. m. per sec. can be handled at a reservoir level of 741·6 m.

Drawings of the main spillway are reproduced in Fig. 21. The spillway tunnel is intended to run free flow at all discharges, and to direct the flow a deflector has been introduced at the bottom of the vertical shaft immediately below which is an air vent. For constructional reasons a second lip has been formed in the roof at the point where the elbow meets the horizontal tunnel.

Much of the interest of the design lies in the fact that the spillway had to be built to fit in with the stages in the construction of the dam. With this in view, the spillway discharge tunnel was first built as a diversion tunnel, following the course indicated in Fig. 16. Upstream of the spot ultimately to be occupied by the elbow of the spillway shaft, an inclined shaft was driven from the roof of the tunnel to emerge at a point on the hillside where a temporary spillway crest was constructed at the initial impounding level

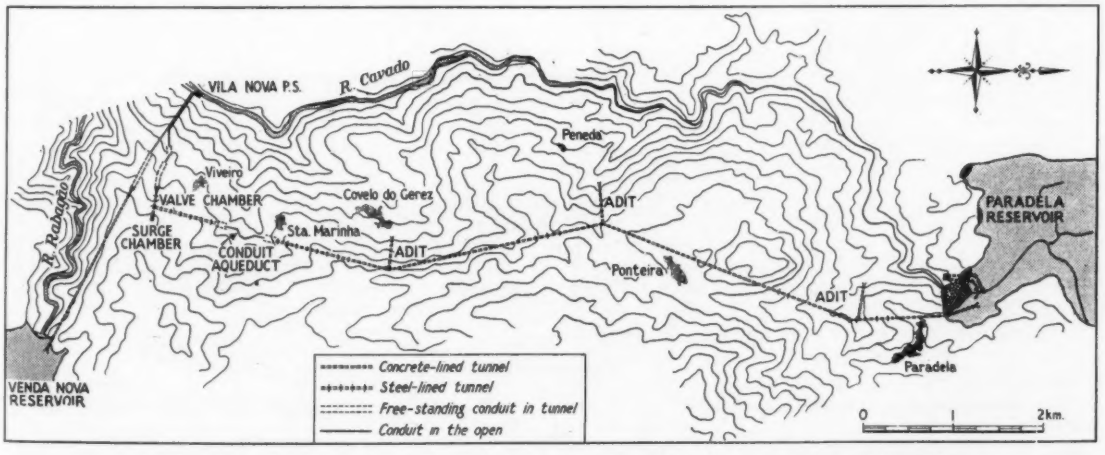

Fig. 22. Map showing the route of the supply tunnel from Paradela to Vila Nova

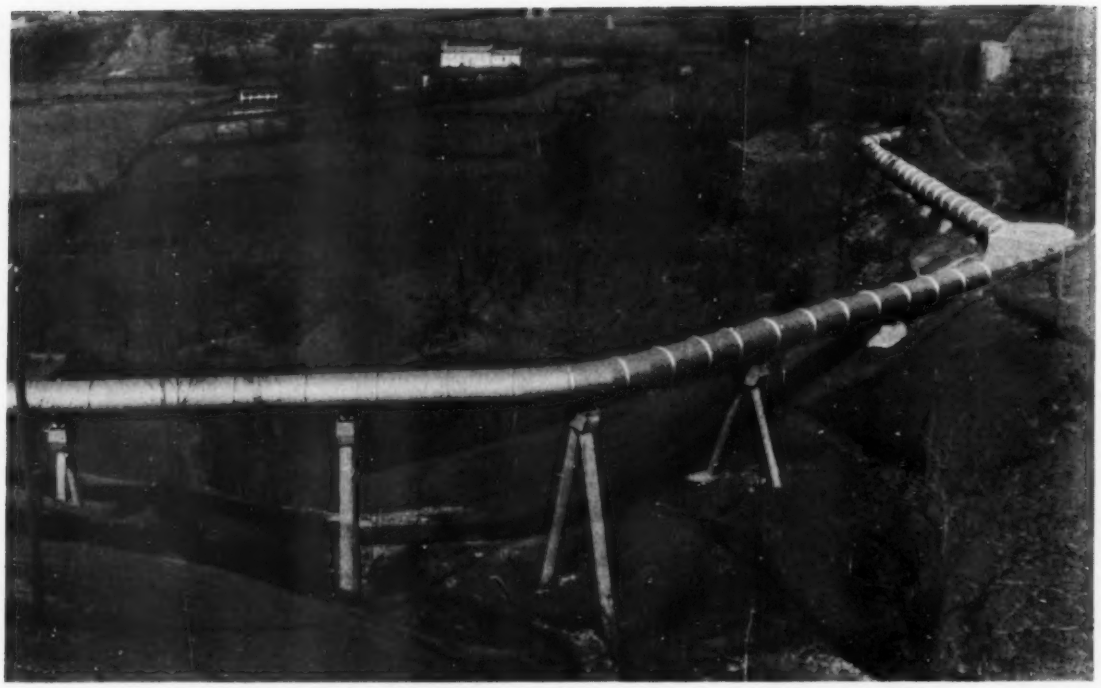

Fig. 23. The section of trestle-mounted steel conduit near Santa Marinha

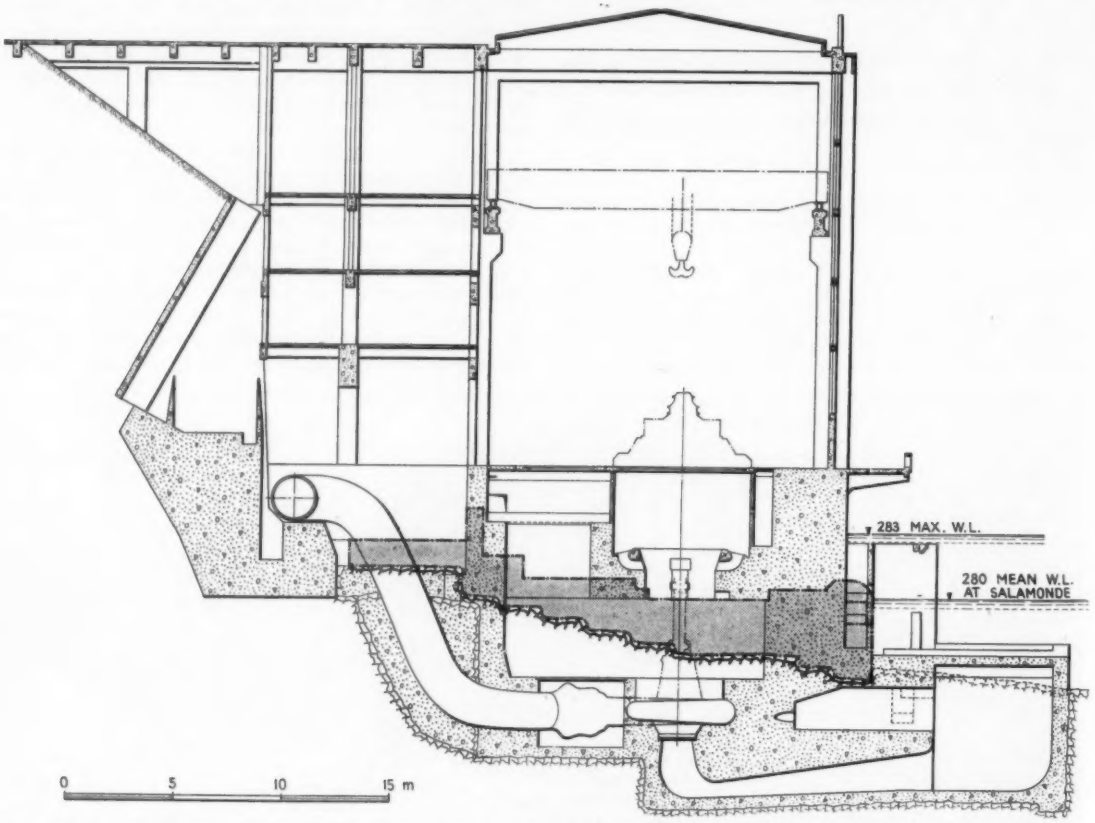

Fig. 24. Section through the concrete structure at Vila Nova for the 60 MVA Francis set. The original concrete for the proposed Pelton set is shown tinted

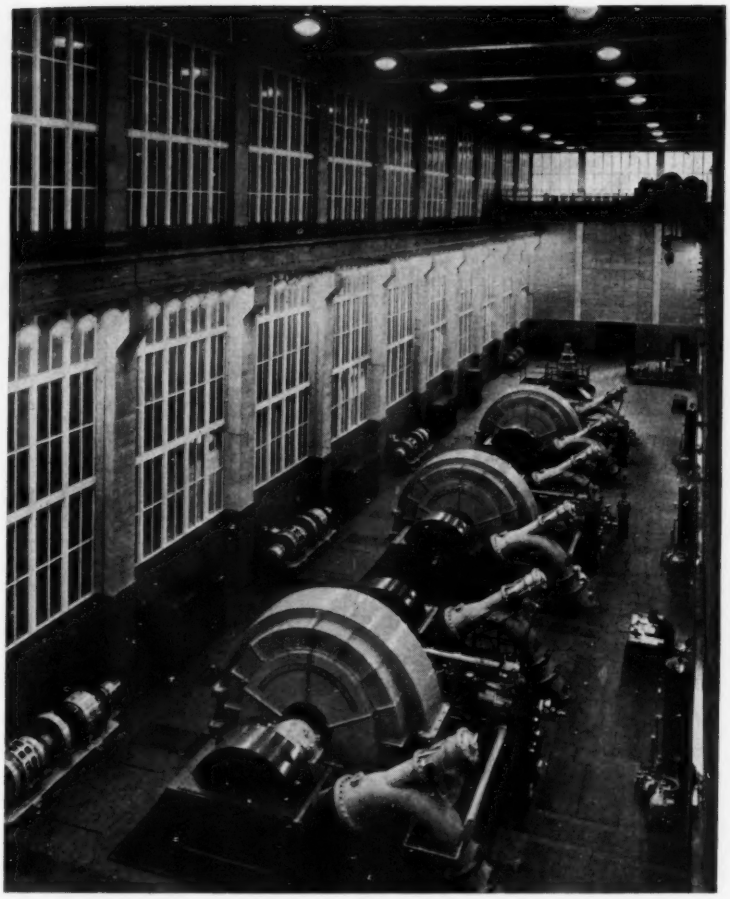

Fig. 25. Vila Nova machine hall showing the new 60 MVA Francis set in the background

of 668 m., and when the dam had reached the necessary height the diversion tunnel was closed upstream of this inclined shaft. This enabled the new set at Vila Nova to operate on a run-of-river basis.

Meanwhile, construction of the spillway shaft proceeded, and was ultimately carried to its full height, but openings were left in the sides of the bellmouth at a height of 725·5 m., as seen in Fig. 21. When stage two of the dam was approaching completion, the temporary spillway was sealed off, the tunnel plugged immediately upstream of the spillway-shaft elbow, and the elbow completed with Prepakt concrete. Impounding was then possible to a height of 725·5 m. (730 m. in a flood) while the dam was completed to full height, after which the openings in the bellmouth were filled in.

The secondary spillway, to which we have referred, is controlled by a sector gate and discharges through a tunnel about 300 m. long to the bed of a small tributary.

Supply Tunnel

The supply tunnel from Paradela reservoir to Vila Nova station, including sections in steel conduit and penstock, is about 11 km. long. A gross head of 460 m. is developed at Vila Nova station, and the tunnel is designed to carry a maximum flow of 24·5 cu. m. per sec.

A map of the tunnel is given in Fig. 22. An unusual feature of the design is that at an intermediate point, near Santa Marinha, the route of the tunnel is followed by a conduit erected in the open for a distance of 660 m. This conduit had to cross a deep watercourse on reinforced concrete trestles, the largest being 16·5 m. high. The conduit is 2·35 m. in diameter and is made of plates 11 and 12 mm. thick. This form of construction proved more economical than carrying the tunnel round the head of this valley. Reasons of topography and also the fact the conduit has no expansion joints made it advisable to abandon the usual straight run of conduit and to introduce a C-shaped bend where it crossed the bed of the ravine. Anchor blocks were introduced at the ends of this bend and also at half length of each straight run between two angles in plan, the bends being left free so that the conduit could expand or contract according to the temperature. To facilitate this the steel saddles carrying the conduit were mounted on the trestles by means of sliding supports permitting transverse movement; longitudinal movements are accommodated by the elasticity of the legs of the trestles. Part of this section is depicted in Fig. 23 of trestle-mounted conduit.

Between the Paradela intake and Santa Marinha there is a length of tunnel 8·2 km. long. For most of its length it is concrete lined and 2·8 m. in diameter, but towards the downstream end, where the rock is less favourable, a thick concrete lining is supplemented by a steel lining 2·25 m. in diameter. Downstream of the 660 m. conduit aqueduct already described is a second tunnel 1·24 km. long. For the first 950 m. it is steel lined similar to the downstream end of the first tunnel, but thereafter the rock becomes too unreliable to maintain a water tunnel, and the construction is changed to a concrete-lined tunnel carrying a free-standing steel pipeline, which, in fact, forms the commencement of the penstock. A valve chamber is located where the steel-lined tunnel gives place to the free-standing pipeline.

Some distance upstream of the valve chamber the tunnel connects with the surge chamber, which consists of an inclined shaft with a lower feed chamber and upper expansion chamber. The upper chamber is in the open and is of concrete.

The penstock section measures 1,326 m. from the valve house to the anchor block at Vila Nova station, and for the last 60 m. it is laid alongside the Venda Nova penstock. Its internal diameter ranges from 2·30 to 2·05 m., and the plate thickness increases progres-

sively from 12 to 35 m. except in the bend in the final anchor block, where plates 43 and 44 mm. thick are used. The steel employed has an ultimate tensile strength of 54 kg. per sq. mm., an elastic limit of 34 kg. per sq. mm. and an elongation of 20%. The permissible working stress was 15·45 kg. per sq. mm. The use of this high-quality material introduced problems in welding which were successfully overcome. The penstock was made entirely in Portugal by SOREFAME, Amadora.

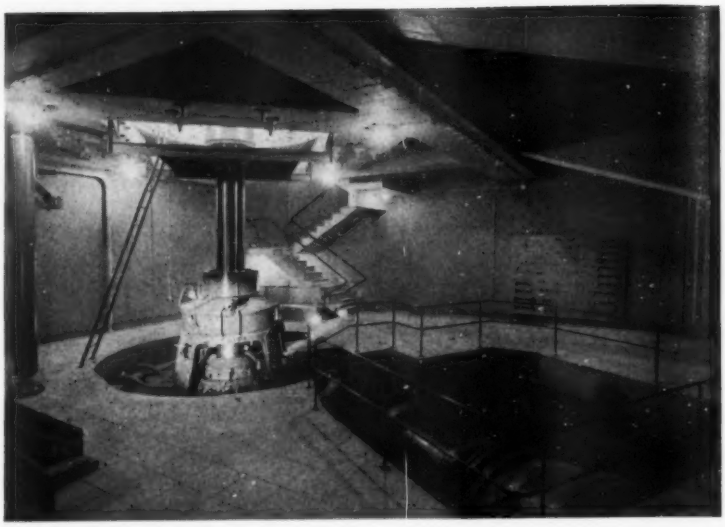

Fig. 26. The open chamber at turbine floor level

Vila Nova Extension

Seeing that we have previously published articles on Vila Nova station, we need only repeat that the first stage, supplied from Venda Nova, consists of three 30 MVA horizontal double-runner Pelton main generating sets, together with an 875 kVA auxiliary set. The turbines, valves and switchgear were supplied by The English Electric Co. Ltd. and the generators by Metropolitan-Vickers Electrical Co. Ltd. The station building is of the surface type, but to obtain the necessary floor area in this steep-sided valley a deep cut had to be made into the toe of the hill, which had to be supported by a heavy retaining wall consisting of buttresses carrying inclined concrete arches. For the same reason of restricted space, the main switchgear is carried on the roof of the station.

When the station was built it was anticipated that the future supply from Paradela reservoir would be handled by a fourth 30 MVA horizontal Pelton set, and space was reserved accordingly, but in the light of new possibilities offered by Paradela reservoir, it became necessary to install a 60 MVA set. Advances in the design of high-head Francis turbines made it possible to select a machine of this type that proved to be feasible and economic.

Although this choice was obviously correct from the power aspect, it introduced several interesting constructional problems, for it involved a deep excavation in the floor of a station that was already in operation, and the structure of which incorporated a heavy retaining wall and was carrying the weight of the main switchgear. This excavation had to be carried about 11 m. below the floor of the tailrace chamber of the originally proposed Pelton set, reaching a point about 7 m. below river bed level. Fig. 24 is a drawing of the concrete constructional work involved, the original Pelton tailrace chamber being superimposed in chain dots and tinted.

One of the problems was to secure one stanchion of the building, because it was necessary to excavate near its footing. This was achieved by sinking two shafts, each 1 m. square, one on each side of the stanchion and filling both shafts with concrete. The tops of the concrete piers thus formed terminated in concrete blocks which embraced the foot of the stanchion in front and rear. The stanchion foot was then secured to these blocks by steel rods passed completely through the assembly and prestressed, after which it was possible to excavate near the stanchion as much as necessary to erect the new penstock. All the walls surrounding the erection pit of the new set were underpinned by alternate sections of Prepakt concrete walls founded at the ultimate foundation level.

Another departure was that instead of supporting the generator on the customary concrete monolith rising from the turbine block, it was decided to carry it on a framework of heavy reinforced-concrete beams about 1 m. wide by 3 m. deep. This was set and excavation of the turbine pit subsequently carried out beneath it. In addition to facilitating the excavation programme, this construction later enabled the erection of turbine and generator to proceed independently, and, as will be seen from Fig. 26, afforded ample room in the turbine basement for subsequent maintenance.

One more problem was to carry the draft tube through to the river. A training wall was built in front of the discharge openings of No. 3 Pelton set to lead the discharge from this set down-river without interfering with the new construction, and a cofferdam was built round the area to be occupied by the draft tube. The draft tube, together with the relief-valve discharge immediately above it, was to be led out through the wall below ground in the space between two stanchions, and to enable this space to be excavated it was spanned by a reinforced Prepakt concrete beam, 1 m. deep by 2 m. wide, constructed in two stages side by side; then when the draft-tube and relief-valve discharge forms had been erected, the space was made good by filling it with solid concrete, which was sealed inside against river seepage with Sika cement mortar. Outside the building the draft tube is curved through a right-angle to a tailrace canal so that it discharges directly outside the training wall previously mentioned. To prevent its becoming obstructed by stones and debris carried down by the river, the tailrace canal is covered throughout its length, and its discharge opening can be sealed by a stoplog.

(Continued on page 56)

Turbine Efficiency Measurement

A summary is given of the better-known methods of determining turbine efficiency for the benefit of the non-specialist engineer

PART TWO

Current Meter

The current meter is widely used in Europe and has many applications for measuring flows of practically every kind, from the gauging of velocity and direction of tidal currents to measurements in the headrace, tailrace, intake or pipeline of a hydro-electric power station. There are two forms of current meter, the propeller type and the cup type, both of which operate on the same principle. However, while the former is universally applicable, the latter is usually confined to river flow investigations. The vanes or cups are mounted on a free-running spindle which is immersed at the required depth in the flow being metered. The impact of the water on the vanes or buckets causes the main spindle on which they are mounted to rotate and the speed of rotation is proportional to the rate of flow at the point where the meter is placed provided it is correctly aligned with the water flow. For the propeller type, the spindle is horizontal and parallel to within ± 5 degrees with the direction of flow, unless a component type meter is used, but for the cup version the spindle is vertical and at right angles to the flow. The cup type of current meter is an adaptation of the anemometer and, indeed, closely resembles this well-known meteorological instrument, but as it is more suitable for use in flows of low velocity and is less affected by inclined flow and fibrous materials in the water, the propeller meter is used almost exclusively for turbine efficiency tests and flow measurements in closed conduits (Fig. 4).

The meter is clamped to a bar and placed in position with its axis in the line of the conduit. The signals emitted by the meter indicate the number of revolutions completed by the propeller and the electrical contacts are so arranged that the time interval elapsing between one signal and the next represents completion of a convenient number of revolutions.

The velocity across a pipe varies, and a single meter would thus have to be traversed across a diameter. To overcome this difficulty, several meters are mounted in the plane of measurement, all running and emitting signals simultaneously. It will be obvious that some form of automatic recording apparatus is essential to deal with the vast number of signals transmitted simultaneously by a set of meters and it has been the practice to use pen recorders to register the impulses both from the meters and from a timing apparatus. This method of recording involves a vast volume of evaluation; in a normal field test, some twenty lines of contact records may have to be counted and correlated to the time intervals, and the final results of a series of tests may therefore only be available after many weeks of hard work. There has been much research to find an accurate method of recording the impulses which also makes available the information required during the actual tests, and at the same time allows a permanent record of the data to be maintained. In this country, the Mechanical Engineering Research Laboratory has evolved a system of electromagnetic counters which record the current meter signals in digitised form. In the case of penstock measurements, this equipment, in conjunction with a novel method of pipe flow integration also developed at the laboratory, enables results to be obtained in a matter of minutes. The current meters are in this case arranged at specified radial positions and results are computed entirely by numerical methods with an accuracy always comparable with, and mostly better than, that of the standard techniques. This apparatus is still in the course of development but, even at this stage, it has overcome some of the major disadvantages of the current-meter method of flow measurement by greatly accelerating the procedure of evaluation without any great increase in cost or quantity of equipment required. In its present form, the apparatus has a separate electromagnetic counter for each current meter; the counters are arranged in vertical columns, each corresponding to a set of current meters mounted at a common radius (Fig. 5). The basic units are the timing standard—which may be either a chronometer or a transistorised frequency standard — an auto-timer and a test unit. A block diagram showing the general arrangement of the automatic digital recorder appears in Fig. 6. The recorder consists of a box fitted with a 35 mm. camera at one end and the counter panel at the other. The inside of the box is painted white to assist lighting and has four car headlamp bulbs in reflectors to illuminate the panel during exposure of the film.

Two types of time standard are available; one is a commercial precision chronometer, the contacts of which close once every second. The alternative unit built at M.E.R.L. is completely transistorised for battery operation and uses a 10 Kc/s. quartz crystal oscillator and decade frequency divider to produce 0·1, 1, 10 and 100 pulses per second.

In contrast to standard recording equipment, where the test duration must be established from a time trace, the M.E.R.L. recorder allows the selection of a time interval, which is thus not only known with an accuracy of $1 \cdot 10^5$ to $1 \cdot 10^6$, depending on which time standard is used, but allows the most suitable choice to be made with regard to specific test conditions and speed of computation. The control unit counts the seconds from the timing standard and allows intervals of 5, 10, 20, 30, 40, 50, 100, 200, 300, 400, 500, 700 and 1,000 sec. to be selected. A second knob allows the interval to be repeated automatically 2, 3, 5, 10, 15, 20 and 24 times. When the control unit receives

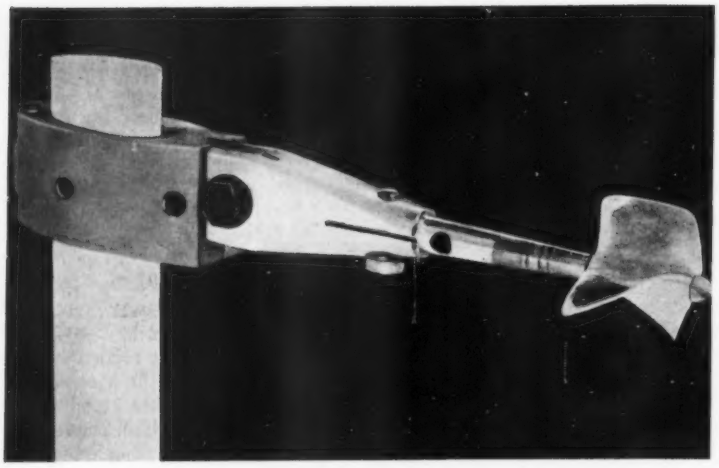

Fig. 4. A typical propeller current meter

the first 1-second pulse from the timing standard, it connects the current meters to their respective counters for the interval selected. At the end of this interval, the control unit:

(a) Illuminates the counter panel;
(b) Operates the recording camera;
(c) Resets counters to zero (if ordered to do so);
(d) Switches off the panel lamps;
(e) Initiates another similar interval of counting if the preselected number of intervals has not been completed, or awaits further operation of the "start" button.

If evaluation is to be carried out in the field, the order to reset counters is cancelled, the recorder box opened after completion of the sequence of events described, and recordings entered in record sheets. In addition to the above operating cycle, the control unit has a counter which can be set to zero at the beginning of the day (or the beginning of a series of tests) and which indicates the number of records the camera has taken. A further counter shows the number of seconds for which the current meter counters have been switched on. These two secondary counters are duplicated on the main counter panel so that their readings are permanently recorded on the negative. From the meter records, the mean velocity through the test section is computed numerically by integration and the use of the meter calibration constants. The standard practice of determining the latter is by towing at uniform speed through the still water of a towing tank, no other mode of calibration giving the required accuracy.

If the standard graphical method of pipe flow integration is used, the velocities of the meters at one radial position, as determined by the current meters mounted at this radius, are replaced by their mean; the product is then plotted against radial position and the area under the curve planimetered. An alternative method consists of plotting the mean velocities against the square of the radius, which again yields the flow through the metering plane. In free surface measurements, for instance in the intake bays of power stations, the mean velocity, and hence the flow, must be obtained by integrating in horizontal and vertical strips over the rectangular plane of measurement.

Current meters are perhaps the best known and most widely used of all methods normally applied for turbine efficiency measurements. Since they are adaptable to many other forms of flow measurement, they have been developed commercially over a period of many years but, for the application covered by this series of articles, they have one important disadvantage—the average meter cannot withstand indefinitely the conditions obtaining in a closed pressure conduit without becoming inaccurate or ceasing to operate altogether, and since the pipe must be opened to remove the apparatus, any delay in recording the data transmitted may compromise a complete series of tests. This limitation does not, of course, exist in free surface measurements with current meters.

Gibson

The Gibson inertia-pressure method of flow measurement, which was developed in the U.S.A. by Dr. N. R. Gibson, is the only method that involves load rejection. It has a peculiar feature in that it takes advantage of a phenomenon which is usually considered with much disfavour in hydraulic engineering, namely water hammer or inertia pressure. Basically, the Gibson method is used to establish the initial rate of flow from a diagram on which are recorded the

Fig. 5. The counter panel

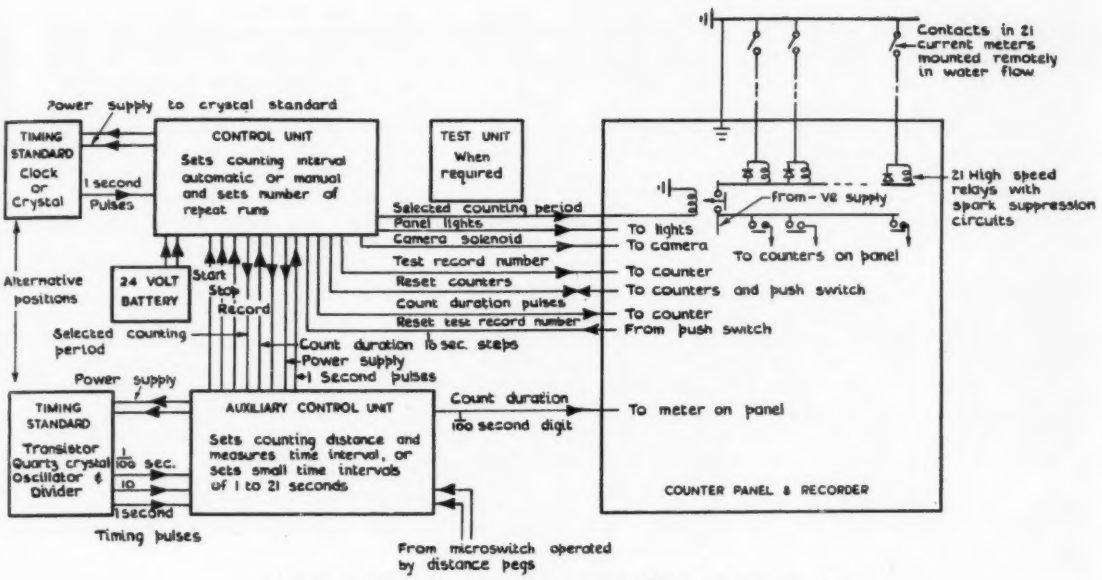

Fig. 6. A block diagram of the automatic digital recorder

variations of pressure occurring in a penstock during shutting down of a turbine in a known time. In other words, the discharge in the turbine supply conduit is deduced from pressure measurements taken during the period in which the regulating gear is closed, thus reversing the procedure usually adopted to calculate the degree of water hammer occurring in pressure pipes. A sketch of the Gibson apparatus is given in Fig. 7.

The conduit, which may be of varying cross-section, is measured accurately to obtain the pipe factor, or the summation of length divided by area. The Gibson method may be applied simply or differentially; for the former, the cross-sectional measurement is taken from the section up to the entrance under a free surface, such as in a forebay or surge tank, and for the latter, from one section to another. When the measurement is at the entrance, the surface level there is recorded so that the diagram obtained from the simple application can be corrected for any rise or fall occurring in the period of shutting down.

The diagram recorded by the apparatus is a photograph of the movement of a mercury column in a calibrated U-tube manometer connected to the conduit by one or both legs, according to the application. The movements are photographed on a sensitive film wound onto a clockwork or electrically driven drum, behind which is a powerful lamp. A pendulum swinging before the camera lens marks time signals on the film, which appear as vertical lines on the final print. The test procedure starts with the turbine running steadily under the required load. After an interval, during which the operating conditions are recorded, the turbine is shut down in one continuous movement. While the machine is being shut down, the load is adjusted accordingly or transferred to other units in the same station. When the gates or the valve are fully closed, the Gibson apparatus is left running for some time to record the oscillations of the mercury column, whereupon the film is removed and developed. Meantime, the turbine is left running idly.

The rate of flow is given by the impulse area of the diagram, which lies between the point of departure of the pressure line from the running line, and the beginning of the first damped oscillation, and is determined on a blue print made from the negative. The line denoting the static level is drawn horizontally and, on the simple diagram, it is corrected for changes in the water level at the conduit entrance. Its height above the running line represents the friction and velocity head that is gradually recovered as the flow is slowed down. The recovery line curves gradually into the static line at the other end of the diagram, and the area above it, suitably corrected for the width of the drum slot, is a measure of the rate of flow that is stopped. Some suitable method is used to measure any flow that has still passed because, even when the turbine gates are fully closed, they may be far from watertight. This can be done, for example, by recording the rate at which the supply conduit empties itself.

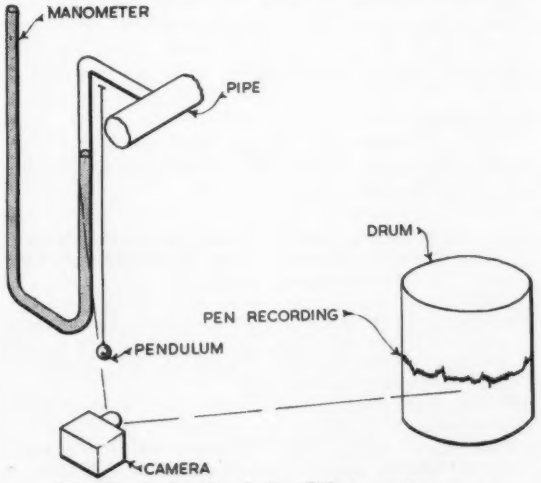

Fig. 7. A sketch of the Gibson apparatus

The ultimate recovery line is found by trial and error. A trial line is drawn and the area on the diagram is divided into vertical strips, the areas of which are added up in succession from the beginning of the diagram; an imaginary area proportional to the leakage through the turbine gates is then added to the final sum to make a total area that would correspond to the initial flow. The ratios of the successive sums of areas of strips to the total area are subtracted from unity; the remainders will be the continuing parts of the initial flow. Friction and velocity heads are proportional to the square of velocity or flow; the remainders are squared and multiplied by the distance from the static line down to the running line at the beginning of the diagram. The resulting lengths are marked off from the static line downwards on the respective verticals separating one strip from the next, and the smooth curve through the ends is the resulting recovery line. The heads recovered are represented by the differences between these lengths and the distance between the static line and the running line at the beginning of the diagram. If this recovery line differs from the trial line, the procedure is repeated until the correct recovery line is found. Finally, the area of the diagram is measured and the net area is obtained by allowing corrections for the effect of the drum slot. The film is marked with gauge points before exposure; the ordinates represent retarding pressure head in feet of water to a scale given by calibration of the manometer and the time scale is given by the intervals recorded on the diagram by the pendulum.

The net area therefore represents the integration of retarding pressure head in relation to time. This is equal to the stopped rate of flow divided by the acceleration due to gravity and multiplied by the pipe factor. Any loss due to leakage is then added to give the total stopped rate of flow.

The Gibson method requires relatively little apparatus and, apart from two tappings into the penstock, its application does not require extensive preliminary work and the conduit does not have to be opened. However, since it involves load rejection, normal power station operation during the tests is impossible and the turbine regulating gear, the penstock itself and the turbine are subjected to very considerable stresses. In addition, evaluation of the results is a somewhat long and arduous process. Various improvements to the recording apparatus have been devised since this method of measurement was originally conceived and it is now generally accepted as an accurate and reliable way of measuring turbine water flow.

(To be continued)

From page 52

As already mentioned, the Paradela penstock is laid alongside that from Venda Nova over the final portion of its run. Within the station, the two conduits continue side by side, and to increase the flexibility of the station a cross connection, controlled by a Louis de Roll valve, enables No. 3 Pelton set to operate either on Venda Nova or on Paradela water.

The Francis turbine for the new 60 MVA main generating set was built by Ateliers des Charmilles S.A. It operates under a gross head of 460 m. and passes 15·5 cu. m. per sec. at full load. The alternator was constructed by Brown Boveri & Co. Ltd., and the associated transformer bank consists of three single-phase units supplied by Siemens-Schuckertwerke, A.G. The circuit breakers, of the ACEC minimum-oil type, were manufactured by SOPREL of Oporto.

Two electrically driven drainage pumps of 50 and 10 litres per sec. capacity are in service, and a petrol-engine-driven emergency pump is to be installed.

Contractors

The civil-engineering design and direction of the works was carried out by Hidro Eléctrica do Cávado, having as contractors:—

Dam, shaft spillway, saddle embankments, and works in Vila Nova power house	Sociedade de Empreitadas de Obras Públicas, Ldª., Oporto.
Temporary diversion tunnel and upstream cofferdam, supply tunnel and secondary spillway	Sociedade de Empreitadas Moniz da Maia & Vaz Guedes, Ldª., Lisbon.
Grouting operations in the dam and spillways ..	Sondagens Ródio, Ldª., Lisbon.
Grouting operations in the supply tunnel	Empresa de Sondagens e Fundações Teixeira Duarte, Ldª., Lisbon.

Acknowledgments

For permission to publish this article we are happy to acknowledge our indebtedness to Hidro Eléctrica do Cávado. Throughout our visit we received the utmost assistance from the engineers of the company's staff, to whom we are particularly grateful for their courtesy and for the trouble they took to supply us with all needful technical information. We also obtained valuable information from two articles in the Portuguese technical journal *Electricidade*: "A Conduta Forçada do Aproveitamento Hidro-Eléctrico de Paradela," by J. Nieto Guimarães and Santos Paupério (April–June, 1957), and "A Cortina Estanque da Barragem de Paradela," by Luís Henrique Gomes Fernandes (July–September 1957).

Transformers for T.V.A. Tennessee Valley Authority have awarded Ferranti Limited a new contract for four 66,667 kVA, 60 c.p.s., single-phase units to form a 200,000 kVA three-phase, 154/69/13·2 kV bank with spare unit. The transformers are for the Alcoa switching station in Tennessee. The order is worth $486,816 and units are required on site by November 1, 1959.

Anodal Insulated Wire. Publication No. O.T.28 issued by the Aluminium Wire & Cable Co. Ltd. fully describes and sets out the advantages of their ANODAL insulated wire. This is a new product and has many uses in modern industry.

Kumagai Gumi Co. Ltd., general contractors, of Tokyo, Japan, have sent us a handsomely produced brochure relative to some of their recent achievements. Established in 1902 this firm has many hydro-electric developments to its credit, among the latest being the Sakuma and Akiba dams and their associated power houses.

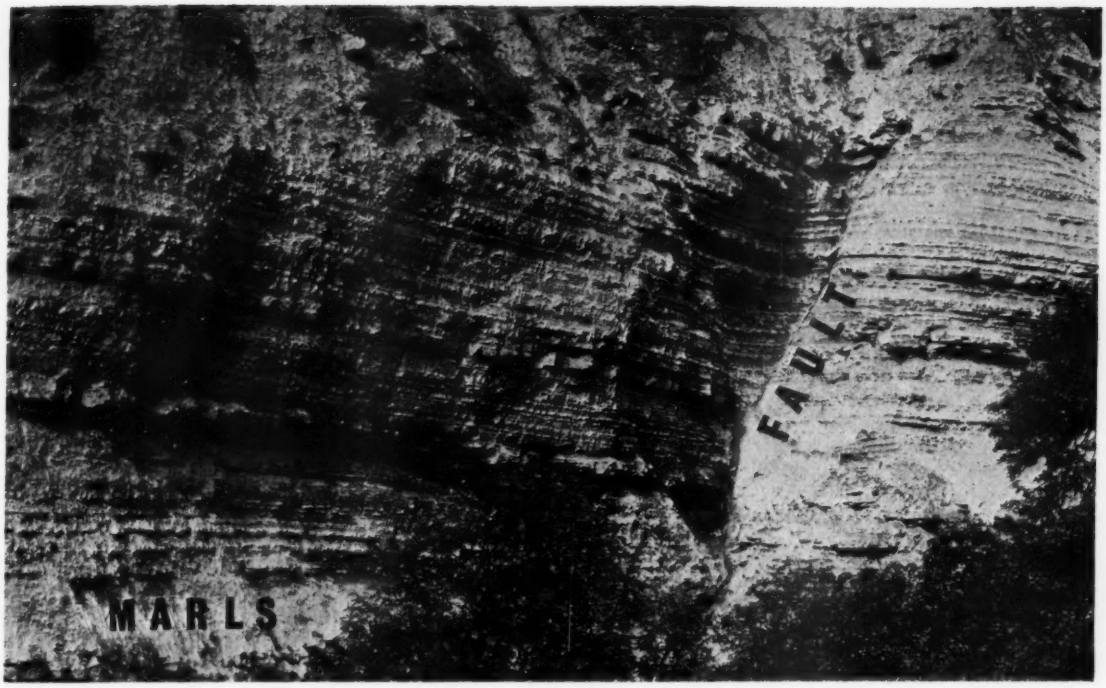

Fig. 1. Calcareous shales and marls with thin beds of argillaceous limestone overlain by boulder clay (Lower Carboniferous Cementstone Group) near Spout of Ballagan, N.E. of Glasgow. Crown copyright; published by permission of the Controller of H.M. Stationery Office

Dam Foundations in Argillaceous Strata

By DR. J. E. RICHEY, F.R.S., F.S.E., F.G.S.,
Consulting Geologist, Dundee

ARGILLACEOUS or clay strata are primarily the concern of geology, one stage removed, as it were, from the province of soil mechanics. Much is known of their physical properties but there is still very much to learn, and it is to be anticipated that valuable results will accrue from researches being pursued in the United States through the combined initiative of geologists and engineers. The main variety of clay strata, shale, may occur in a stratigraphical succession in large thicknesses and so be the only rock encountered at a dam. More often, perhaps, the rock is interbedded with layers of sandstone, or limestone, and varies in amount in different geological venues from thick beds to mere films separating beds of the associated arenaceous or calcareous rocks.

Shales and sandstones are complementary to one another in regard to the two chief qualities of rocks required for dam foundations, namely, impermeability and strength. Shales are usually impermeable though lacking in strength, and may be compressible under the weight imposed by the dam. Sandstones are usually adequately strong, but are in general permeable and water carriers. Repeated alternations of these two types are of common occurrence in many stratal successions and the engineer has often utilised their contrasted qualities in selecting his foundations. The situation is especially favourable when the strata are inclined up-valley, both for purposes of strength and for inhibiting leakage below the centre of a dam and at its abutments. These conditions are present, for example, at the proposed Ramganga Reservoir, in the Garhwal District of the United Provinces, India.[1] The rocks consist mainly of alternating groups of sporadically-cemented porous sandstone and uncemented clay-shale strata belonging to the middle and upper Siwalik Beds of Tertiary age, and have an inclination (dip) of about 30° upstream. The shales will be included in the proposed dam foundations and should prevent leakage through the sandstone. An earthen structure with clay-core inclined upstream, 400 ft. in height, is planned, and the clay-core will abut against one at least of the up-valley dipping beds of clay-shale.

In contrast, horizontal bedding or down-stream inclination at low angles of interbedded sandstone and shale strata may prove dangerous where insetting of

the base of the dam into the rock is inadequate. A frequently quoted instance is the Austin Dam in Pennsylvania, U.S.A., 50 ft. in height, which failed in 1911 through slippage of sandstone on shale[2], where the toe of the dam was not inset into rock.

Classification of Argillaceous Strata

Shale is consolidated mud which was deposited in thin layers of slightly varying composition, termed laminae. It is in consequence a fissile rock, splitting more or less easily along the planes of lamination. Mudstone is similar in composition to shale, but differs structurally in that bedding planes are set at wider intervals and may be obscure or absent; it is thus deficient in well-marked planes of easy splitting. Mud, the detritus forming both types, may be described as clay essentially composed of clay minerals with a varying amount of fine sand, which is mostly quartz (silica). According to the grain-size of the sand certain main varieties of shale and mudstone are distinguished:

	Rock types	
Composition	*Fissile*	*Non-fissile*
Essentially clay ...	Shale	Mudstone
Clay mixed with very fine sand	Silty shale	Siltstone
Clay mixed with fine to medium sand ...	Sandy shale	Sandy mudstone

The above geological classification is of value to the engineer, especially as a greater amount of sand produces a stronger rock. A fourth type of clay rock results from the addition of calcareous material to muds (Fig. 1). The calcareous part is usually calcium carbonate, the rock-type being calcareous shale and, in the case of mudstone, marl or marlstone. When calcium magnesium carbonate is present as the calcareous material the rocks are termed dolomitic shales and dolomitic marls or mudstones; or when iron carbonate or iron oxides are dominant constituents, ferruginous shales and mudstones. Under certain arid conditions of formation, hydrous calcium sulphate is deposited from solution along with mud which, when consolidated, forms gypsiferous shales and mudstones.

Consolidation of Muds

The degree of consolidation of argillaceous rocks is of especial importance to the engineer. Two stages in the process are compaction and cementation. Interstitial water is held by capillarity between the tiny mineral particles of muds, and its expulsion leads to *compaction*. Muds may contain upwards of 50 per cent. of water and this is driven off by pressure due to the weight of overlying later-deposited sediments, or, at a still later geological stage, to folding of the strata. *Cementation* results from the tiny mineral particles becoming pressed together at their contacts when much of the water has been driven off. The particles are then welded together by the crystallisation of mineral matter along their opposed surfaces. This produces much stronger as well as more durable rocks than if the strata are only compacted. The contrast between compacted and cemented shales and mudstones is especially evident when the rocks are alternately wetted and dried. An engineer may, indeed, use this rough method in order to determine the physical durability of a clay rock in which he is interested. The compacted kinds soon revert to mud (slaking) or scale off in flakes (spalling). Partly cemented kinds deteriorate only after repeated soakings and dryings. Cemented kinds remain unaltered.

Weathering

Compacted shales and mudstones may be affected extensively by weathering, and where near-surface outcrops occur fairly deep excavation down to unaltered rock is often entailed. The brassy-coloured iron sulphide minerals, either pyrites or perhaps the more easily altered magnetic pyrites, are prevalent as tiny particles in clay strata, in the dark grey but not the red coloured varieties. Their decomposition by rain water leads to the formation of weak sulphuric acid, and this in turn hastens the decomposition of the rock. Even with cemented or partly cemented kinds there is a marked contrast between the durability of shales and mudstones and well consolidated arenaceous strata. An example from the North-West Highlands of Scotland is quoted here, where quartzite is the durable though well jointed arenacous type, and the less durable rocks, cemented dolomitic shales (so-called Fucoid Beds) and an overlying porous sandstone, the Serpulite Grit, all being of Cambrian age (Fig. 2).

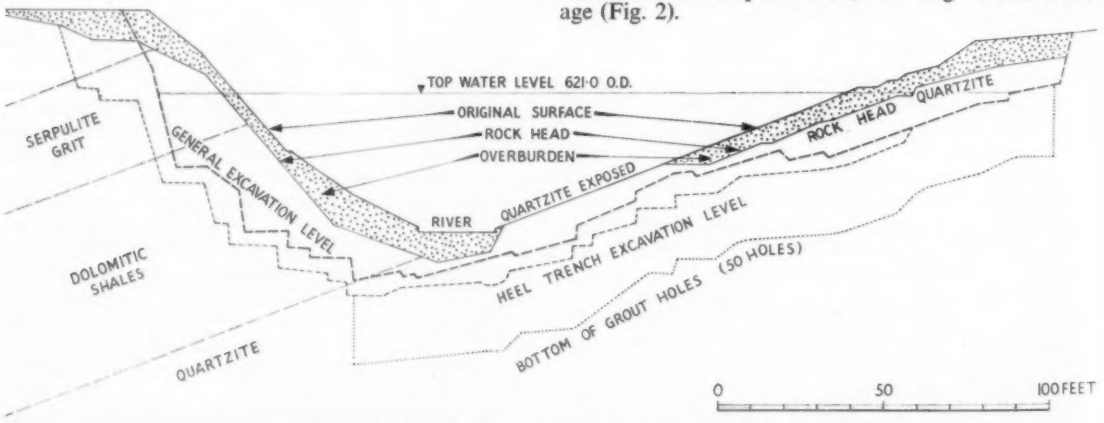

Fig. 2. Section in Cambrian strata, Loch Dubh, Ross-shire, looking upstream. Published by courtesy of Messrs. Blyth & Blyth, consulting civil engineers, and The North of Scotland Hydro-Electric Board

Calcareous and dolomitic argillaceous strata are usually better consolidated than the normal kinds, with cementation effected by the crystallisation of the carbonates they contain. On the other hand, calcareous shales are prone to be deeply decomposed owing to the removal of the calcium carbonate by percolating rainwater. Dolomitic varieties are more durable, as calcium magnesium carbonate is not easily soluble. Gypsiferous shales and mudstones are readily broken down, owing to the solubility of hydrous calcium sulphate (gypsum). The occurrence, known or suspected, of any gypsiferous strata at or below a dam-site is therefore a matter for especially close investigation.

There are also many instances of clay strata showing little decomposition even in environments where difficulties might have been expected. A notable case is that of the Gyobyu dam in Burma, some details of which may be given. The site is in a river gorge in the Pegu Yoma hills about 55 miles north of Rangoon, the dam being an earthen structure 134 ft. in height and 700 ft. long, with a concrete core-wall. The strata, mainly a "blue shale," belong to the Middle Tertiary (Oligocene-Miocene) period, and the series of rocks concerned were upheaved only a few million years ago to form the youngest and lowest mountain-range in Burma. The shale proved easy to excavate.[3] For example, it was removed in a river-diversion tunnel situated at some depth below the site almost entirely with pneumatic picks. It would appear to have been merely compacted, and at any rate cannot have been well cemented. The bedding is at a high angle and almost directly upstream. No untoward slippages of the shale took place during excavation, nor does weathering or spalling of the rock appear to have given any trouble. This seems all the more remarkable because construction was necessarily prolonged owing to the country being overrun by the Japanese. The shale was only slightly permeable, and any possible leakage was provided against by the construction of a cut-off wall some 15 ft. in depth below the river-bed and still deeper along either abutment where the shale was found to be less sound.

Where exposed in an excavation, uncemented shales or mudstones usually deteriorate rapidly either through repeated wetting by rain, or merely by drying out and so contracting. Freshly exposed surfaces therefore require protection with a layer of asphalt or cement, or the situation may have to be dealt with in some other way. As an example the procedure adopted in dealing with a thick bed of red siltstone at the Conchas dam in New Mexico may be cited.[4] The siltstone, 64 ft. in average thickness, was exposed along the lower part of the steep sides of a canyon on the South Canadian river (Fig. 3). Especially on the north side spalling proved to be a major difficulty and was met by excavating a trench clear of the adjacent canyon wall down to an underlying pink shaly sandstone, and erecting a monolith of concrete at this place. The interval between this initial monolith and the canyon wall was then dealt with by continuing the procedure, with a final monolith poured in direct contact with the vertical face of the siltstone which had been finally cleaned.

Expansion and Contraction

Argillaceous strata, the uncemented varieties more especially, are liable to elastic rebound which may lead to cracking when overlying load is removed in an excavation. Under the weight of a dam uncemented types are also liable to be compressed and contract. Allowances to be made for compaction may be considerable, and earth or rock-fill dams are usually erected in these cases. One exception was the Howden Masonry dam in the Derwent Valley in Derbyshire, 117 ft. in height above bed of river. The strata consisted mainly of Carboniferous shales, and were found to be only slightly compressible, with settlement of only 1 inch in 5 years where the excavation was deepest.[5] At the Conchas Dam, already referred to, the arrangement of the strata across the site involved the slow compaction of an upper red siltstone, estimated as 4 in., with differential settlement of the dam. A central, concrete portion was founded on pink shaly sandstone underlying the siltstone or on the still lower upper artesian sandstone. Compaction of a lower red siltstone proved by borings was not anticipated. On the other hand compaction had to be allowed for in the case of the upper red siltstone below the canyon sandstone, and earthen portions were therefore constructed on either wing, with a joint separating each wing from the central concrete portion.

In cases where shale or mudstone of great thickness is concerned, greater amounts of consolidation have been recorded. An example is a thick group of shales of Early Tertiary age in the Missouri valley, U.S.A., which are famous in another connection for their wealth of small primitive mammals. The shales were found to consolidate as much as 2 ft. under a load of approximately 16 tons per sq. ft.[6] An instructive case from Algeria is described in a valuable work recently published on the geology of dams by Professors Gignoux and Barbier of Grenoble.[7] This is the Ghrib dam near Algiers 70 m. high, which is founded on Middle Tertiary (Miocene) sandstone overlying thick shales and with alternating beds of sandstone

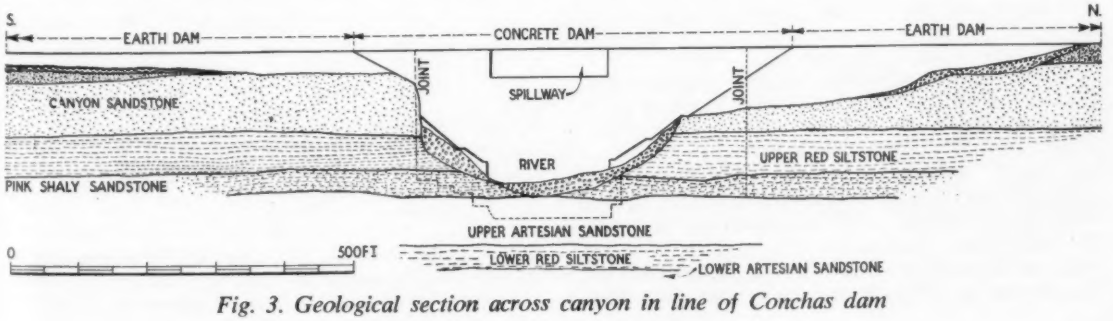

Fig. 3. Geological section across canyon in line of Conchas dam

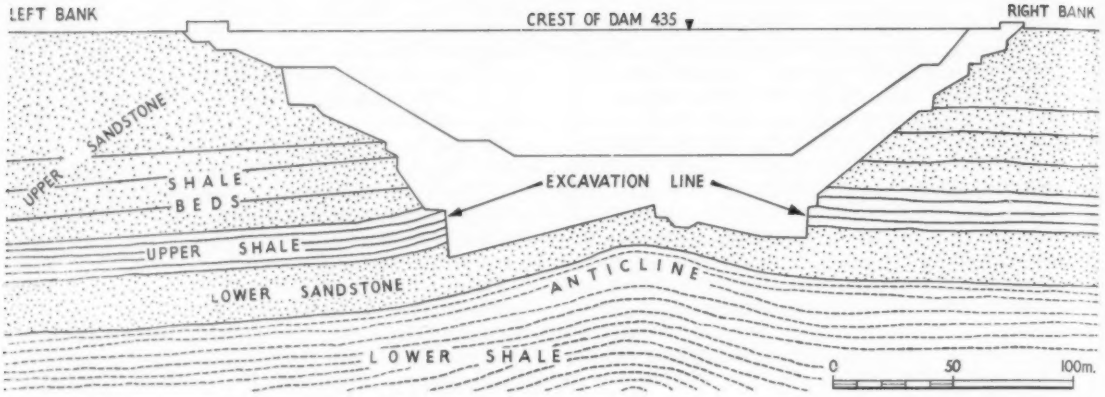

Fig. 4. Geological section of Ghrib dam site, Algeria, showing compressible shales and permeable sandstones (Middle Tertiary)—after Gignoux and Barbier; published by Mason & Co., Paris

and shale forming the two abutments (Fig. 4). The pressures exerted on the foundations by the rock-fill dam and by the water in the reservoir produced a depression of the level of the bed-rock amounting to 30 cm. (1 ft.). Part of the compression took place fairly gradually during the building of the dam (years 1933–35, see Fig. 5) and part, with special suddenness, on the filling of the reservoir in the year 1939. It will be noted on Fig. 4 that at the abutments of the dam a considerable amount of shale together with sandstone overlying the lower sandstone was excavated, in order to allow the central part of the dam to be founded directly on the lower sandstone which as a rock was practically incompressible. Its crushing strength was found on test to be 180 kg./cm.², whereas that of the shales was only 7 kg./cm².

Gravity Effects on Slopes

The comparative weakness of argillaceous rocks leads to the development of breaks and displacements along the sides of valleys, which are less usual with stronger strata. A major kind of displacement, namely landslips, are however found in all types of rock.

Landslips. In poorly compacted types of argillaceous strata gravity displacement may occur very slowly owing to viscous flow; and this may eventually lead to the development of fracturing and so to sliding *en masse* which, as in other cases, may be very rapid and disastrous. Recent landslips about which so much has been written are easily enough recognised by the tumbled kind of topography characteristic of them. In the case of ancient slips all surface irregularities may have been removed by denudation, and their occurrence beneath a valley-floor may be discovered only when a site comes to be excavated, though they may have been detected, or suspected, during exploration by boring.

An especially complicated section of ancient landslides of shale and sandstone was exposed in the trench for the Woodhead dam (No. 2) on the River Etherow in the Longdendale valley east of Manchester (Fig. 6). The original irregular surface of the landslipped masses had been planed off by valley denudation, and indications were lacking at the surface of the difficulties entailed for the engineer, although many of these were known from a prolonged boring programme as well as from a nearby dam (Woodhead No. 1 dam) which had failed[8]. Some excavation, however, was saved by driving a heading in the disturbed strata on the valley side, with its roof near to top-water level of the reservoir, for the construction of a cut-off wall. From the section of the trench reproduced in Fig. 6 it will be evident that in similar cases boring should disclose discrepancies in an anticipated disposition and succession of the strata and go far to assist the engineer or the geologist in his assessment of a site.

Minor Displacements. From the point of view of the engineer, "insidious" would be the most fitting epithet for a variety of small displacements which have been encountered in argillaceous rocks at many dam sites, for they are often not known to occur, unless by extensive boring, prior to excavation. Several of the instances to be cited are situated in the northern half of England where compacted or poorly cemented shales of Carboniferous age are prevalent strata. Repeated fracturing of argillaceous rocks along valley-

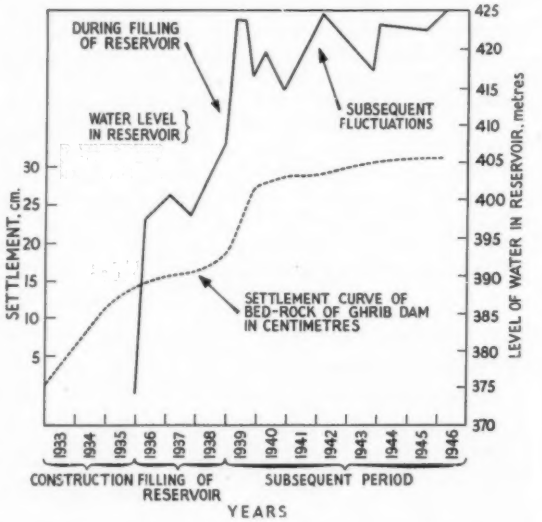

Fig. 5. Ghrib rockfill dam, Algeria. Settlement curve of bedrock in relation to construction of dams, filling of reservoir and subsequently (after G. Cheylan)

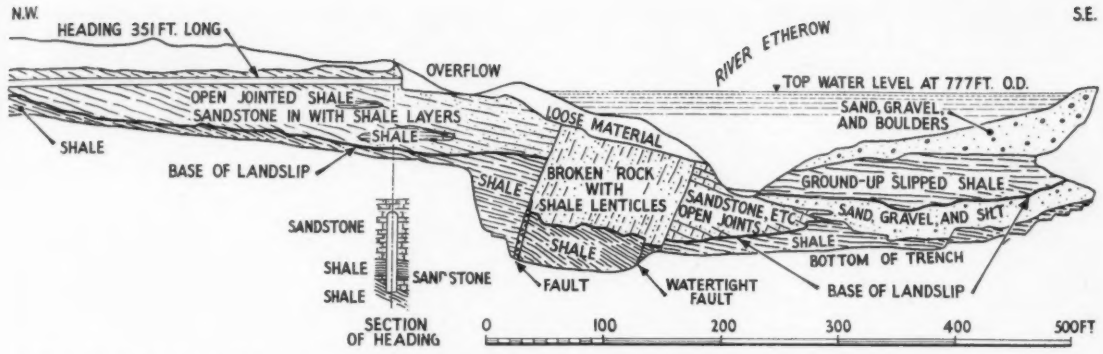

Fig. 6. Ancient landslip exposed in trench excavated for Woodhead earth dam (after J. F. Bateman)

sides, disturbance of strata along valley-floors owing to pressures exerted by unstable valley-slopes, bulging of strata in excavations, and rock-creep, are the main effects which have given trouble during the planning and construction of dams.

Extensive cracks parallel to the steep sides of a canyon were discovered during exploration of the site for the Mount Morris dam on the Genessee river, New York[9]. They traversed Devonian shales reported to be of excellent quality, and persisted inwards for a distance of 100 ft. from the canyon sides, being in some instances open fissures 6 in. in width (Fig. 7). In addition, horizontal shear planes had developed in the horizontally bedded strata. All these effects were ascribed to the action of gravity, and the engineers realised that three special difficulties were involved. Water would escape from the reservoir along the cracks. The water would be under hydrostatic pressure and this would probably weaken the wall rocks of the canyon just below the dam and might lead to their collapse. At abutments the pressure would reduce the capacity of resistance of the strata to horizontal forces. In these circumstances it was considered advisable to construct a concrete cut-off wall to the full width of the fractured belt of shale.

A similar difficulty occurred at the site of the Derwent dam across the River Derwent in Derbyshire, England[5]. This is a masonry structure 114 ft. in height from the river. The rocks consist of shales with thin layers of sandstone belonging to the lower part of the Millstone Grit formation which overlies the Carboniferous Limestone. On the valley sides numerous extensive cracks were found traversing the shales and sandstone, and as in the Mount Morris case the cracks were inclined steeply on each side towards the river at angles of about 60° (Fig. 9). They were undoubtedly the result of the valley sides settling, under the influence of gravity, in the direction of the river. The valley sides, however, were very much gentler slopes than in the Mount Morris case, and the widths of both belts of fractures very much greater, being about 650 ft. on the western side of the valley. Excavation had to be taken to the full width of the fractured belts.

Also in the Derwent valley, shales with thin layers of sandstone were found to be rucked up in the form of an abrupt anticlinal fold at the centre of the valley, the disturbance terminating abruptly in depth. At the Howden dam site this depth was about 120 ft. (Fig. 8). The strata on either side of the ruck were much broken though not folded. Similar rucking was encountered at the Derwent dam already referred to (Fig. 9). A theoretical explanation of the rucking which those interested may wish to consult was given by P. M. Croswaite in the Discussion appended to the paper referred to. Another instance was disclosed in the trench for a dam in the North of England, namely, the Hury dam on the river Balder (Fig. 10). There the shales, which are interbedded with thicker sandstone intercalations than in the Derwent valley, were highly disturbed below a deep cover of glacially deposited boulder clay. The site was noteworthy for another reason. Shales were exposed in the bed of the river, but rock-head was found to be inclined in both directions away from the river. This unusual rock-head configuration has also been discovered at Lough Dan north-west of Glendalough, Co. Wicklow, Ireland, as the writer was informed by Mr. MacDonald, Chief Engineer, Electricity Supply Board, Dublin. Downstream of the Lough the outlet river flows on rock but the banks are of glacial moraine. Exploration by boring for a proposed dam led to the discovery that the rock-head inclined away from the river beneath the moraine on either side, and that the river-course, by some strange chance, happened to coincide with an ancient local watershed.

A related effect to rucking along valley floors is the bulging of incompetent strata, notably shales on the floor of excavations. Nothing comparable with the immense 30-ft. high bulges which occurred in the floor of the Culebra cut of the Panama Canal has

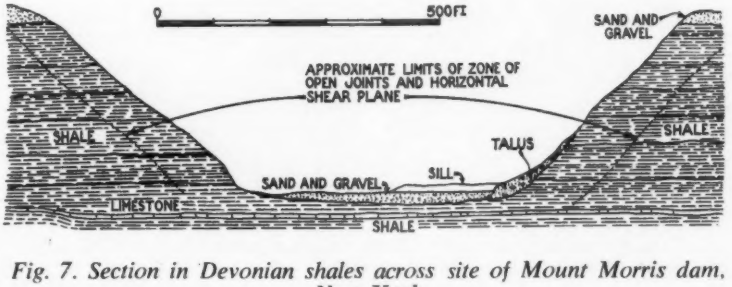

Fig. 7. Section in Devonian shales across site of Mount Morris dam, New York

Fig. 8. Contortions in shales and sandstones of Lower Millstone Grit age in puddle trench of Howden dam, Derbyshire. The disturbances die out in depth and are absent at the bottom of the trench, 120 ft. below the surface. Geological survey photo by Sir Jethro Teall; reproduced by permission of the Controller of H.M. Stationery Office

ever been recorded from a dam site, as far as the writer is aware.

Rock creep on the sides of valleys is another phenomenon which is likely to occur with steeply inclined, thinly bedded rocks such as shales, or with their metamorphic equivalents, slates and mica-schists. Especially when superficially weakened by weathering, the outcropping ends or "heads" of the beds become bent over by the pull of gravity, and fracture is likely to occur along the plane of the bends. The effect is sometimes superficial, and sometimes deep-seated. Instances from Alpine valleys were recorded by Lugeon where the disturbance extended more than 100 m. in from the surface, measured horizontally, and more than 50 m. vertically. It is to be suspected where strata with highly inclined bedding extend parallel to steep valley sides. A geologist can usually detect it, not necessarily from exposures along side streams, cutting downwards to the undisturbed rock, but from his knowledge of the geological structure of the adjacent country with which the apparent anticline does not fit. In cases where the disturbances are extensive in depth, a site is indeed difficult and may be impracticable. At a dam site at Pinet on the Tarn, in France, quoted by Lugeon (Fig. 11), the procedure he recommended and which was adopted was the excavation of terraces with great care by stages down the slope, with each step in turn supported by a wall of concrete which was extended in a trench downwards to and into undisturbed strata. This revet-

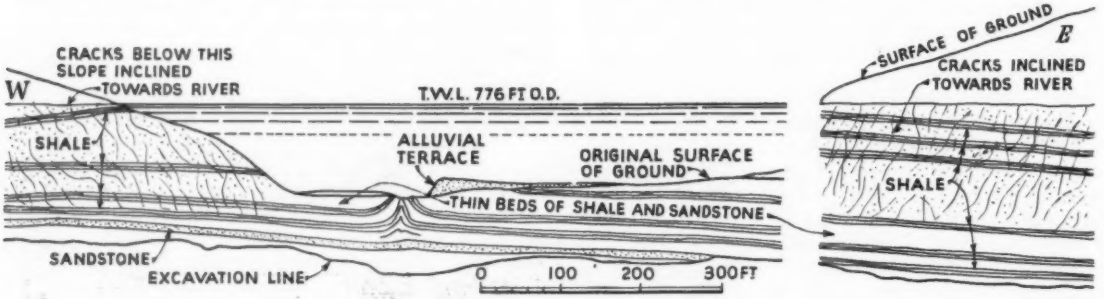

Fig. 9. Gravity creep of valley sides causing superficial rucking up of shales of Lower Millstone Grit age at centre of valley and cracks at sides inclined towards the valley floor, Derwent dam site, Derbyshire, England. After E. Sandeman, Proceedings Inst. C.E. 206, 1920

ment stabilised the slope and finally the deformed rock was rendered water-tight and secure by grouting.

Turn-over of the heads of beds at outcrops or below thin coverings of earth is also caused by the drag of moving ice and is of common occurrence in Scotland and no doubt in other recently glaciated countries.

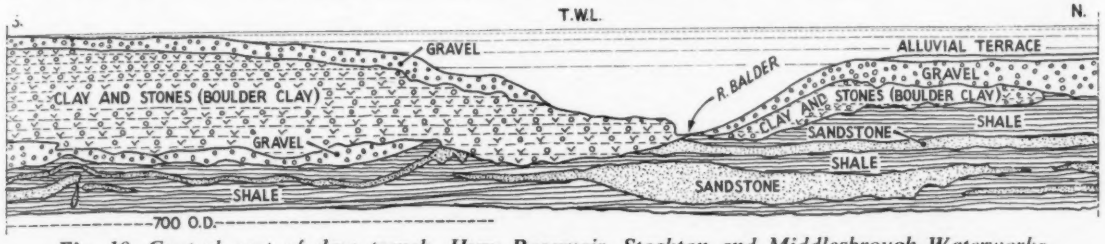

Fig. 10. Central part of dam trench, Hury Reservoir, Stockton and Middlesbrough Waterworks

Uplift Pressure

It is fitting to recall that the well-known difficulty of hydrostatic uplift pressure at the base of a dam was first appreciated and allowed for, strangely enough, in respect of argillaceous strata. The rock may be itself impermeable, but in the firmer, cemented kinds bedding planes may be open and joints also, and through these water may penetrate. Such argillaceous rocks occurred at the site of the great Vrynwy masonry dam in N. Central Wales, which was constructed about the year 1886. One of the consultant engineers, Dr. G. F. Deacon, showed outstanding geological insight in this matter and also in the siting of the dam, as he realised the possibility of uplift pressure in these rocks and incorporated drains in his structure to give relief[8].

Faults

It has been already mentioned that a favourable feature of shales and mudstones is their impermeability as materials. Faults that traverse argillaceous strata, too, are usually impermeable. In most cases the strata comminuted by the movement along the fault-plane form an impermeable though soft layer along the fault plane, known as clay gouge. Even in strata consisting of alternations of sandstone and shale, broken sandstone fragments in the fault vees, that is, the space between the two solid walls of a fault, are usually set in a matrix of clay gouge derived from the shales, and again the fault is impermeable. While faults of this kind may not present water difficulties where they cross a dam site, they form highly inclined belts of weakness in the foundations. On the other hand there have been cases in argillaceous rocks where fault vees have been filled with hard though broken material. An example comes from the Claerwen concrete-gravity dam in Central Wales. There the rocks consist of shales with steep bedding planes striking across the site, and were traversed by a fault with vees some 7 to 8 ft. in width at rock-head. The fault-rock in the vees consisted of fragmented material with a clay matrix which was impervious and also strong. In fact, it was so well consolidated that its excavation entailed pneumatic drilling[10]. In contrast, peeling-off of the shale itself in the excavation caused trouble.

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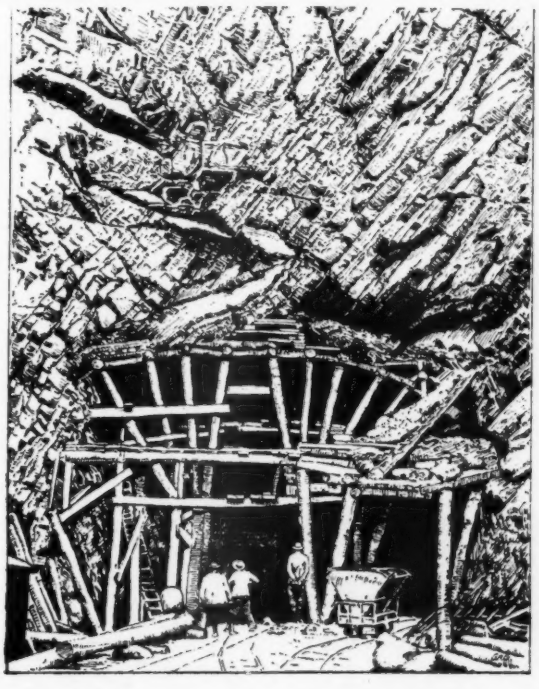


Fig. 11. Inclined and broken strata due to rock creep at entrance of river diversion tunnel, Pinet-sur-le-Tarn, France (drawing by G. Goodlet, after M. Lugeon in "Barrages et Geologie," Rouge et Cie, Lausanne, 1933

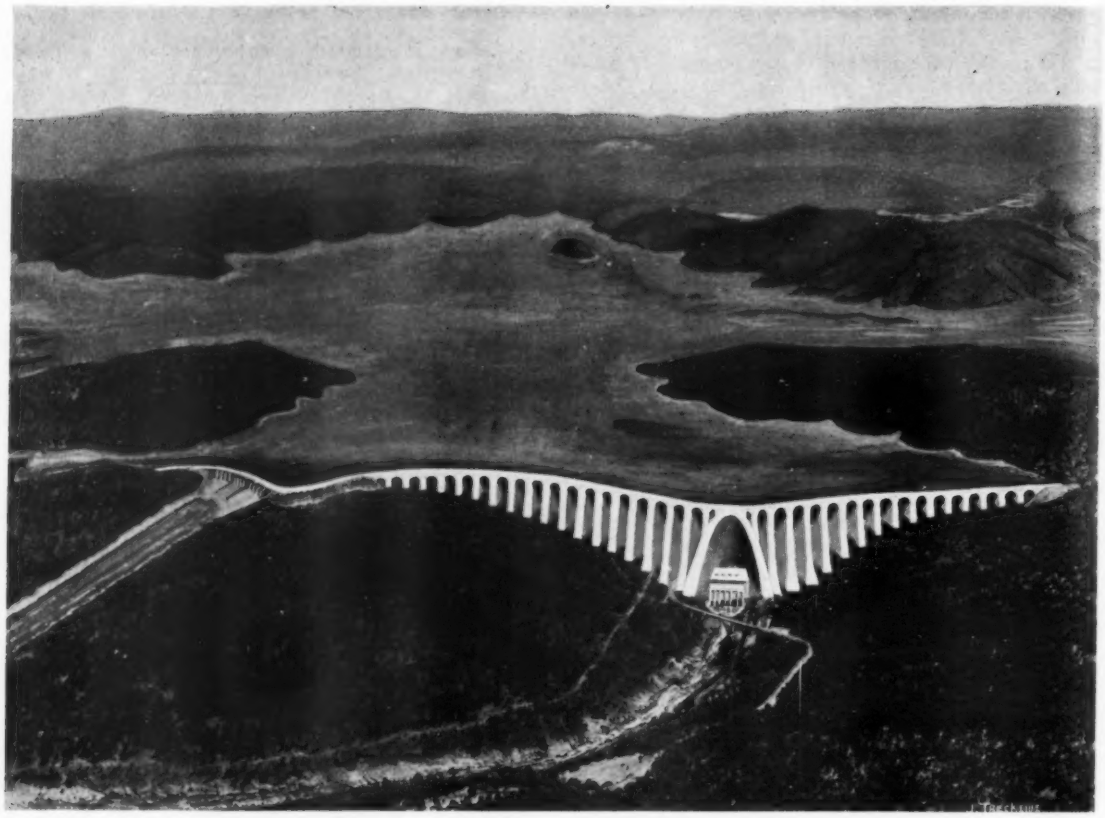


Fig. 1. A sketch showing the proposed Oroville dam

Feather River Project

An account is given of this interesting Californian scheme which is designed to secure maximum economy in the use of water for a variety of purposes

*By IAN B. MACKINTOSH, M.A., A.M.I.C.E., A.M.ASCE.

THE sharp contrast between the Sahara-like deserts surrounding the Los Angeles Metropolitan area of Southern California and the luxuriant sub-tropical vegetation inside gives the impression that the area itself enjoys an abundant rainfall. This vegetation is, however, dependent on continuous irrigation. The rainfall averages 15 in. per annum at Los Angeles and 10 in. at the Beaches. The population of the five counties has grown from one quarter of a million in 1900, through one million in 1920, three and a half millions in 1940, over six millions in 1954, and estimates predict a population of just short of ten millions in 1975. The *per capita* use for water is also growing and it is to provide for the increased needs that the California Water Plan, of which the Feather river project is the initial unit, has been conceived.

Until the 1970's, supplies of water to Southern California are assured from Colorado river sources 220 miles away, but thereafter there will be a shortage. To meet this shortage the Feather river project provides for the delivery of water from the Delta region near San Francisco to Southern California beginning in 1976. The project, which is estimated to cost $1,500 million, is before the State Legislature for approval.

Oroville Dam

The proposed Oroville dam, shown in Fig. 2, will, if built, be the highest dam in the United States. It would create a reservoir with a gross capacity of 3·5

* Senior Engineer, Taylor Woodrow Construction Ltd.

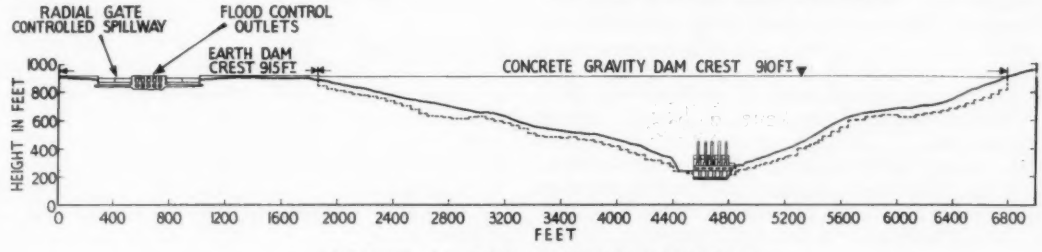
million acre-ft., of which 2 million acre-ft. would be drawn on to provide seasonal and cyclic regulation. At the dam site, the Feather river has an average annual run-off of nearly 4 million acre-ft. There is also additional storage upstream of the site of nearly a million acre-ft., owned by electric and water utilities. The Oroville power plant, located at the foot of the dam, would have a capacity of 440 MW.

The purpose of the Oroville installation is:—

(1) To earn money for the project by the sale of power.

(2) To supplement flows in the Sacramento-San Joachim Delta to provide a steady supply there for water export to areas of deficiency.

Flood control and irrigation will also be provided for in the Feather river area itself.



PROFILE OF DAM LOOKING UPSTREAM

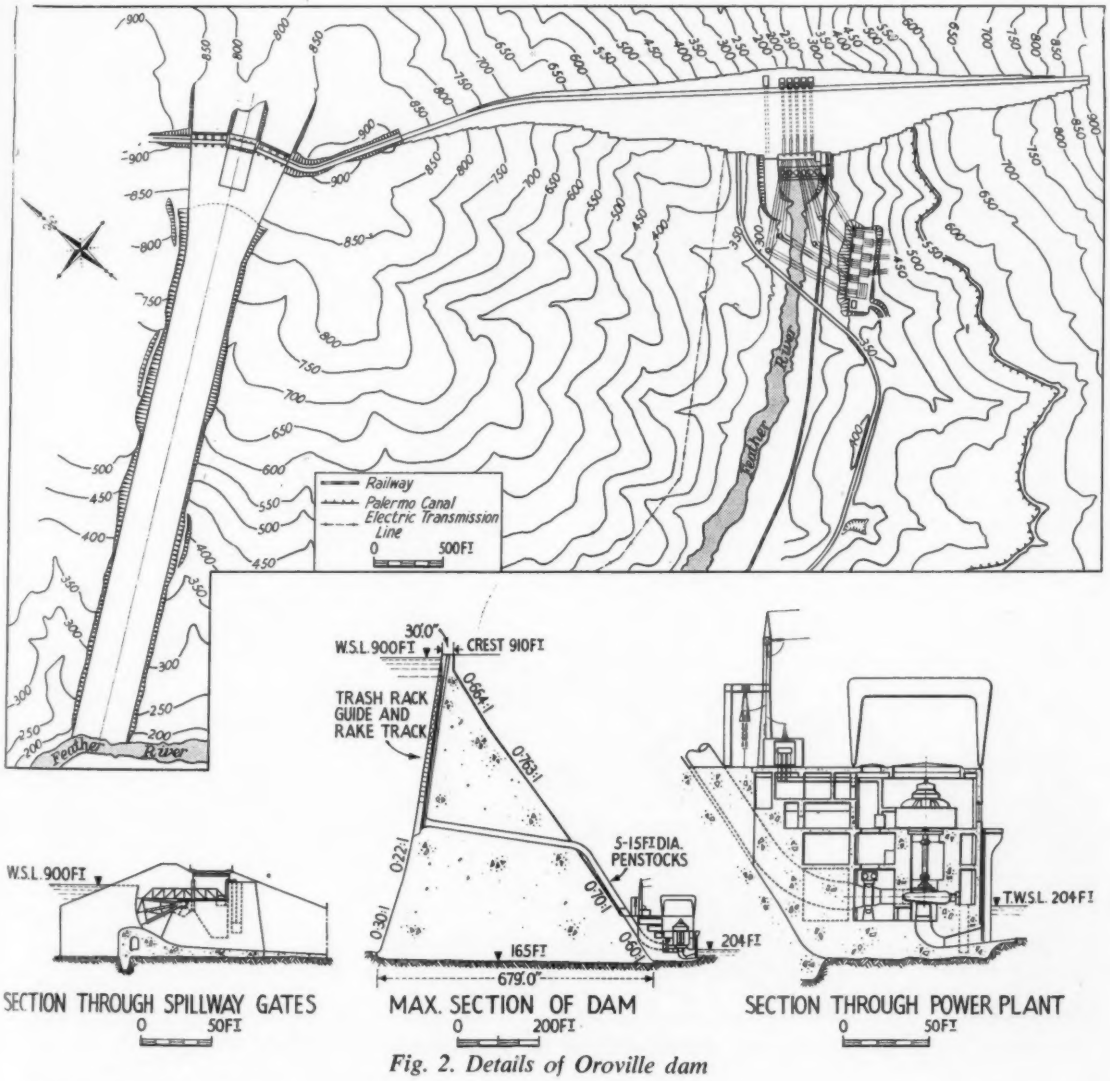


Fig. 2. Details of Oroville dam

The main proposal for the dam, shown in Fig. 2, is that it should be of the mass-concrete gravity type, 720 ft. high, containing 14 million cu. yards of concrete, compared with 10½ million cu. yards at Grand Coulee dam. An alternative proposal may be considered in rockfill, although hitherto a dam approaching this height is yet to be built, the highest earth dam being the Swift dam in Washington, at present under construction, which is 520 ft. high.

Design studies are at present being made on other types of structure, namely an arch-buttress dam, which is represented by the artist's sketch in Fig. 1, and a more or less conventional multiple-arch dam. No decision has yet been made on the type of dam to be adopted.

The layout included in the 1955 report contemplated the construction of afterbay dams in the channel of the Feather river downstream of the main dam for the purpose of re-regulating the river. Further studies indicated that the capacity of these afterbays would be inadequate for complete re-regulation and a scheme similar to that in Fig. 3 was developed. As will be noted, the re-regulating storage would be provided in a reservoir completely off the stream of the Feather river. Water would be conducted to this reservoir through a canal on which an additional power drop of approximately 90 ft. would be developed.

Pumping Costs

Since the need for pumping would, under full-project operation, be five times as great as the gross power output of the Oroville plant, it will be necessary to earn the maximum amount for the power by selling it at its highest value, that is, for peak load, to offset the heavy cost of pumping. This will be done by selling the entire output of the Oroville plant to the Pacific Gas and Electric Company and purchasing

Fig. 3. Proposed scheme for the Feather river development

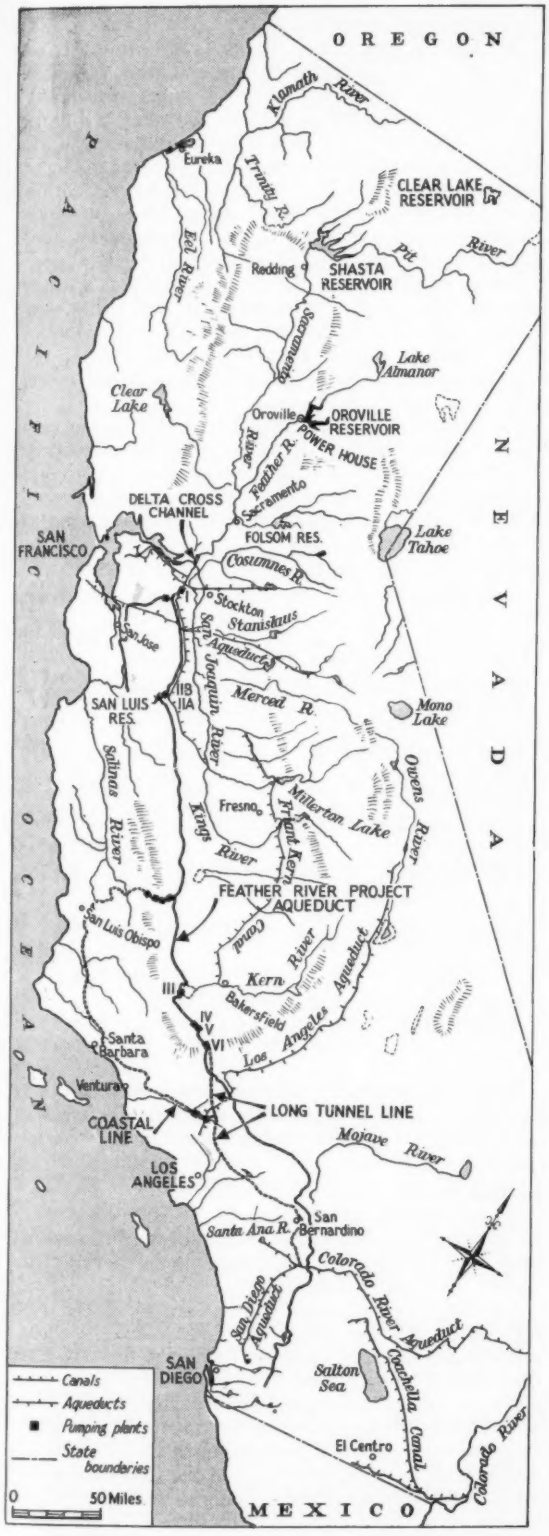


Fig. 4
Map showing main features of project

off-peak power to operate the pumps. This means that the installed capacity of the pumps will have to be twice that of the average output, and further that storage at each pumping lift must be provided if the canals themselves are to have a capacity that is not materially greater than the average flow. The annual charges on the increased capital cost of the pumps and some of the conduit due to the doubling of the capacity, would be considerably more than offset by the reduction in the cost of power for pumping.

The High Line Route

The map in Fig. 4 shows the main features of the project. Water would be passed down the river from the Oroville reservoir into the Sacramento river, a few miles from the mouth of which a proportion of the flow would be transferred into the San Joachim Delta by means of the Delta Cross Channel. From there it would be conveyed by an unlined channel from headwater in the Delta to pumping plant No. 1 near Tracy. The water would be lifted from about sea level to the Feather River Aqueduct, at about elevation 230 ft., which would parallel the Delta-Mendota canal of the Central Valley project to San Luis Creek. The concrete-lined canal would deliver water by gravity into the San Luis forebay which would extend upstream to two pumping plants at the foot of San Luis dam, named pumping plants IIA and IIB. One plant could lift either directly into the project canal at elevation 402 or into the San Luis reservoir and the other plant would lift only into the reservoir.

San Luis dam would be of the earthfill type, 310 ft. high, containing 68 million cu. yards of material and would create a reservoir with a storage of a little over two million acre-ft., the whole of which capacity would be utilised. This storage would:—

(1) Re-regulate pumped water so that a variable monthly demand for irrigation purposes in Fresno County could be satisfied, as well as an irrigation demand in Kings and Kern Counties, in which there would be a two-month no-irrigation demand, and a ten-month uniform irrigation demand.

(2) Supplement the cyclic storage provided for the project as a whole more economically than at Oroville where loss of head, resulting in a consequent loss of high-value power, occurs when the reservoir is drawn down.

The project conduit would follow on grade contour along the west side of the San Joachim valley, a distance of about 264 miles, and discharge into the Buena Vista forebay. There, pumping plant III is located providing a lift to elevation 500 ft., and thence to pumping plants IV and V, which are separated by 1½ miles on Wheeler Ridge. From the discharge of pumping plant No. V, at elevation 1,500 ft., the conduit continues to pumping plant No. VI in Pastoria Creek. This plant would lift the water to elevation 3,357 to the inlet portal of two consecutive tunnels, totalling 10½ miles in length. These would convey the water through the Tehachapi mountains to Quail Lake afterbay.

The Quail Lake afterbay would re-regulate the water to provide a continuous flow to the south. The conduit, after extending along the south side of the Antelope Valley, would cross the Los Angeles aqueduct and then pass over the Mojave desert in an easterly direction. It would then follow a series of

tunnels and a siphon to a spillway into Horsethief Canyon, on a tributary of Pine Valley Creek, which leads into Barrett Reservoir in San Diego County.

This is the terminal of the Feather river project aqueduct at a distance measured along the channels of 740 miles from Oroville dam. The total length of the conduit starting from the point of diversion in the Sacramento river would be about 585 miles. A branch of the main channel would take off just above pumping plant No. I, and by a series of canals and short tunnels, would serve the Santa Clara Valley and San Benito County. The total length of this branch would be about 30 miles.

THE HIGH LINE ROUTE

Distance miles	Feature	Elev. feet	Conduit capacity cusecs
0	Oroville dam	200	
	Sacramento river		
150	Pumping plant No. I ...	0	
		230	11,000
224	San Luis forebay	190	
	Pumping plants IIA & IIB	402	7,000
320	Kings County line ...	340	
413	Buena Vista forebay ...	310	4,000
	Pumping plant No. III ...	500	
			6,000
441	Pumping plant No. IV ...	490	
		800	5,000
443	Pumping plant No. V ...	790	
		1,500	5,000
458	Pastoria creek	1,480	
	Pumping plant No. VI ...	3,357	5,000
472	Quail Lake afterbay ...	3,330	
			2,000
596	Mojave river	3,240	
			1,100
638	Riverside County	3,100	
			800
660	San Jacinto river	2,860	
			500
698	San Luis Rey river ...	2,760	
			300
718	San Diego river	2,700	
			200
738	Horsethief canyon ...	2,600	
	Barrett reservoir		

The conduit capacity between the Buena Vista forebay and the Quail Lake afterbay is double that required to pass the average flow at a uniform rate to enable off-peak power to be utilised for pumping.

The route described is the one recommended in the report to the Legislature and it is known as the High Line Route. Other possible routes are the Coastal Line and the Long Tunnel Line, neither of which are directly comparable because they do not feed quite the same areas as the High Line Route.

Long Tunnel Line

The principal feature of this is a 27-mile-long tunnel through the Tehachapi mountains which would require the water to be raised only to elevation 1,870 ft., that is, the elevation of the tunnel, compared with elevation 3,357 ft. in the High Line Route.

The report states:— "The 20 ft. diameter tunnel would pass through a very active seismic region that has experienced several major earthquakes within recent historic times. Six major faults would have to be crossed, including the longest and second longest in the State which intersect about 8 miles west of the tunnel, a situation which in itself portends the existence of fractured rock and general tunnelling difficulties. The existence of more than 160 springs in the vicinity of the tunnel line indicates that heavy inflows of water are to be expected whenever faults or zones of fractured rock are encountered. Ground water may often be under considerable pressure due to the great thickness of cover over the tunnel. Two hot springs exist near the south portal location, indicating that drainage and air conditioning problems will probably be encountered in that area. Pockets of natural gas may be trapped within the sedimentary formations to be tunnelled."

The tunnels would be constructed from three permanent access shafts ranging in depth from 1,600 ft. to 1,800 ft., as well as from the portals. An estimate has been made of the cost of constructing the tunnel in which eminent consulting engineers and tunnel contractors co-operated.

The estimated cost of the whole tunnel was $357,466,000, made up as follows:—

	Millions of $
Construction facilities and utilities ...	1·34
Sinking three shafts	3·63
Excavation	83·45
Timbering	12·25
Steel tunnel supports	28·36
Concrete lining	40·97
Grouting	6·88
Tunnel ventilation	1·78
Dewatering	3·59
Camps and maintenance of roads ...	3·89
	186·14
Contractors' overhead and profit (33·6%)	62·78
	248·92
Engineering and contingencies (25%) ...	62·23
Interest during construction (15%) ...	46·31
	$357·46

Based on a six-day working week the tunnel is estimated to take eight years to construct. This works out at an average weekly advance of about 77 ft.

Further Power

There are two modifications to the High Line route which would provide power plants south of the mountains and develop the head built up in carrying the water up into the mountains.

The first of these would have a forebay on the west fork of the Mojave River near the inlet of a three-mile tunnel to Devil Canyon. From the tunnel outlet the conduit would extend to a power drop of 1,600 ft. giving an ultimate installed plant capacity of 440,000 kW. Thence the water would be conveyed to an afterbay reservoir near San Bernardino. Part of the water would have to be re-lifted to feed the Colorado River aqueduct at the San Jacinto Tunnel portal.

The other modification is to terminate the main conduit at Quail Lake. Part of the water would be distributed locally from this point and the remainder put through a series of tunnels and a 700 ft. drop to Liebre

Gulch power plant. From there the water would pass through more tunnels to a reservoir on Castaic Creek where there would be a plant developing a head of 1,100 ft. The ultimate installed capacity of both plants would be 516,000 kW. The earthfill dam on Castaic Creek would be some 200 ft. high and provide a storage capacity of 100,000 acre ft.

In both cases the plants would be operated in the peak hours of commercial demand in the region and develop the maximum dependable power capacity in order to realise maximum power revenue to assist the pumping bill. Both the Coastal lines and the Long Tunnel line could be made to terminate in the reservoir on Castaic Creek.

In 1956 the Legislature directed that additional studies be made of alternative coastal routes. Extensive investigations are still in progress and no conclusions have yet been reached.

In addition to the above, attention has been given recently to the possibilities of combining pumped storage features developing peaking power with the pumping lifts along the line of the conduit. Again no definite conclusions have been reached.

In 1957 the Legislature appropriated $25 million for construction work in connection with highway and railway relocation near the Oroville dam site. Two railway tunnels and two railway bridges are included.

Values of On Peak and Off Peak Power

As stated earlier all power produced by project plants would be sold to the Pacific Gas and Electric Company as peak power, and power for pumping at off peak periods would be purchased from that company. P.G. & E. have indicated what the prices of the power purchased and sold would be, based on 1954 values.

A thermal station located in the San Francisco Bay area was assumed as the source of power at a capital cost of $125 per kW and oil at $1·85 per barrel.

The value of power to P.G. & E. delivered at 120 kV at Oroville would be:—

(1) Dependable capacity per kW year $20·83
(2) Non-dependable capacity per kW year ... ... $ 4·20
(3) Usable energy per kWh ... ... 2·73 mills.

Non-dependable capacity, i.e., capacity in excess of dependable, must be known in advance to be available for continuous periods of at least six months duration.

The cost to the State of power for pumping would be:—

	Delivered to pumping plants numbered		
	1 and 2	3, 4 & 5	6
(1) Peak power Cost per kW per month	$2·50	$2·80	$2·90
(2) Off-peak power Cost per kW per month	$0·125	$0·325	$0·35
(3) Energy cost per kWh ($1 = 1,000 mills.)	3 mills.	3 mills.	3 mills.
(4) Distance from Bay area in miles	80	290	320

Peak power is related to the highest kW demand in a 12 month period. Off-peak power would be scheduled by the Company so as to pump an average weekly flow of water specified by the State within the limits of a specified maximum demand. Pumping capacity would have to be not less than twice the average pumping capacity required to take advantage of off-peak energy.

In interpreting the above prices it should be borne in mind that a company like the Pacific Gas and Electric Company pays Federal and local taxes to the amount 25 to 30 per cent. of gross annual revenue, and that these taxes raise the gross annual charges against a capital investment to about 12% of the capital sum expended.

Cost Estimates

Oroville reservoir, dam and power plant	$418,661,000
High Line Route including San Benito branch	$1,174,247,000
Total ...	$1,592,908,000
Cost of Oroville works and Coastal Line to San Jacinto tunnel portal	$1,837,567,000
Cost of Oroville works and Long Tunnel Line to San Jacinto tunnel portal	$1,638,252,000

Acknowledgments

The author's thanks are due to the State Engineer and the Director of the Department of Water Resources, State of California, for providing information on this project.

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(2) The Metropolitan Water District of Southern California—History and First Annual Report 1939.
(3) Ditto—16th Annual Report 1954.

Data-Processing System. A brochure issued by Ferranti Limited describes their Pegasus system of data processing, which is claimed to be fast, versatile and reliable.

Electricity Production in Uruguay. We have received from the General Direction of the State-owned Electricity Generating Plants and Telephones, No. 4, April 1958, of their home journal UTE. This elegant publication analyses the main features of the financial year 1957 and summarises the statistics of production and consumption with the help of excellent graphs. The balance sheets of hydro production for 1956 and 1957 appear on pp. 16–17. The last pages of the issue are devoted to the Rincon de Baygoria plant, and include impressive illustrations of the progress of the work.

Dominion Engineering Co. Ltd. The highlights of the 1957 production of the Dominion Engineering Co. Ltd., Montreal, are displayed in a special issue of their home journal "Dominion Engineer," Vol. XXV. The place of honour is given on the front cover to one of the 45,000 h.p. 57 ft. Kaplan runners supplied to the order of the New Brunswick Electric Power Commission for the Beechwood power station. Among the other items included in the pages devoted to the Hydraulic Division of the Company, we should like to single out the illustrations of the nozzle branch and the workshop assembly of a manifold for the 150,000 h.p., 2,800 ft. head impulse turbines of Alcan's Kemano plant.

Silt Erosion in Turbines and its Prevention

Y. K. Murthy, B.E., M.S., M.Eng.(Harvard), A.M.I.E. (India), and K. Madhavan, B.E., discuss the problem of silt erosion in turbines and the design of works to remove silt from water fed to hydro-electric stations

HYDRAULIC turbines are sensitive to the effects of cavitation, corrosion and erosion. Regarding cavitation, a great deal of experimental work has been done both at the research laboratories of the manufacturers of hydraulic turbines and at various research institutes. Considerable progress has been made towards developing methods of eliminating the damage due to cavitation. Corrosion has also been limited to a great extent by the use of stainless steel for runners.

Serious erosion effects, however, are being experienced in many installations where proper precautions are not taken, and efficiency is adversely affected. In this article, an attempt is made to discuss some aspects of silt erosion and its prevention.

Effects of Erosion on Efficiency

Wear due to cavitation, corrosion and erosion can be distinguished by close examination of the worn-out surfaces. Wear due to cavitation is indicated by craters and depressions in the turbine blades as if particles of the material have been picked out piece by piece, whereas the wear due to corrosion is indicated by flaky brown scales formed on the surfaces. Erosion by silt is indicated by roughening of surfaces, and by deep cuts and grooves as if the material has been gouged out by a chisel. Among the three types of turbines, namely, Francis, Kaplan and Pelton, the last is the most sensitive to erosion effects. This is because the roughened surfaces of the nozzle and needle tend to distort the impinging jet and cause it to lose its compactness. The impinging jet breaks into spray and so loses a part of its kinetic energy. In addition to this, the splitter on the bucket, being a sharp and delicate edge, will also get damaged and create further loss in efficiency. It has been estimated that even 0·5 mm. to 1 mm. wear of the needle can produce as much as 9% loss in efficiency[1].

In Francis turbines, as the relative velocity of the water in the runner is low, the effect of any roughness is not so detrimental. The silt that is passing through the clearances, however, will wear the runner band and the stationary ring, thus increasing the clearances and reducing efficiency.

In the case of Kaplan turbines, the effect of erosion is not very important, except when large amounts of heavy silt particles pass through the runner and cause erosion on the periphery of the blades, especially at the inflow edge.

The maximum size of particle that can be safely permitted to enter is the main criterion in the design of silt-exclusion works. By experience gained in the working of the existing plants and evaluating the damage caused by erosion, it has been found that the maximum particle size which can safely be permitted will vary between 0·1 mm. and 0·7 mm., depending on the type of turbine, viz:—

Pelton wheel	0·1 mm.
Francis turbine	0·3 to 0·5 mm.
Kaplan	0·5 to 0·7 mm.

Other factors which determine the choice of the limits of the particle size are:— *a* mineralogical composition of the silt, *b* clearance between moving and stationary parts, and *c* material of the runner.

Composition of Silt. If the silt is composed of hard material like quartz, the particle size should be limited to a low value. In the absence of hard material the design may allow for a larger particle size.

Clearance Between Moving and Stationary Ring. The clearance between the runner and the stationary discharge ring is an important consideration in the case of Francis and Kaplan turbines. The size of silt that can be permitted to pass through the turbines depends upon this clearance. However, if the turbine has to be specially designed for operation in waters heavily laden with silt, the turbines have to be fitted with replaceable discharge rings, which in some plants are made of rubber. In some cases, devices for the injection of clean water into the clearances are provided. When this is so, a less-exacting design of desilting basin is permitted.

Material of the Runner. If the runner is made of stainless steel, it can withstand wear better, and can be repaired easily by welding. On the other hand, materials like bronze wear out more easily and have to be replaced more frequently.

Measures for Excluding Silt

To protect plant using silty waters and to minimise the wear on the turbines, the measures adopted can be classed as follows:—

(i) Proper location and design of intake works.
(ii) Desilting basins and silt excluders constructed in the water-conduit system.

Proper designing of the silt-exclusion arrangements is especially important in run-of-river plants. During the major part of the year, almost the entire flow of the river is diverted for power generation, and the excess of flow available for silt exclusion is limited.

Location and Design of Intake Works. The positioning of the intake should be such that as far as possible the rolling bed silt should be prevented from entering into the water conduit system. In a bend of the river, the outer curve of the bend tends to get scoured, whereas deposition takes place on the inner curve. The levels of water on the outsides of the bends

are slightly higher than on the insides due to centrifugal action. A spiral flow is thus caused (Fig. 1). The velocities are also higher on the outer side because of the greater depth and lessened friction. Near the latter half of the bend, on the inside, there is usually a region of almost still water which facilitates sedimentation, so it is important to avoid locating an intake on the inner side of a bend unless the topography of the site may warrant otherwise. As far as possible, the intake should be located on the outer side of the bend.

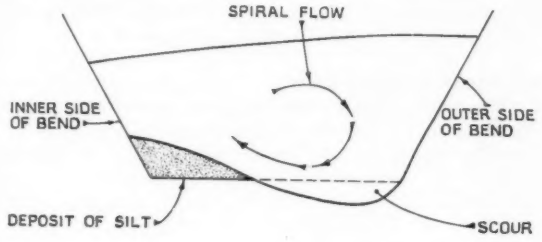

Fig. 1. Formation of a spiral flow

The most general procedure for excluding rolling material entering the water-conduit system is to have the sill level of the intake higher than the bed of the river. In many canal intakes on rivers in India, however, it has been found that this measure by itself does not suffice for efficient performance. Deposited material banks up in front of the intakes and the rolling material is carried over into the canal. In the Nangal barrage, the intake for the power canal has been protected by concrete ducts which were designed in the light of extensive model studies[2]. The general arrangement is shown in Fig. 2.

The Zurich Federal Institute, Switzerland, have developed another effective arrangement for the exclusion of silt, as shown in Fig. 3. This arrangement is in use at the Morel and Lavey hydro-electric schemes[3, 4]. In the Lavey scheme the main barrage across the river Rhône consists of three spans. Normally, two of these are able to handle the maximum flood discharge of the river. The other span, *S*1, between the intake and guide wall is normally closed and is operated for disposing of the floods when either of the other two gates is inoperative. The water level between the guide wall and the intake becomes higher than that of the flowing water in the other

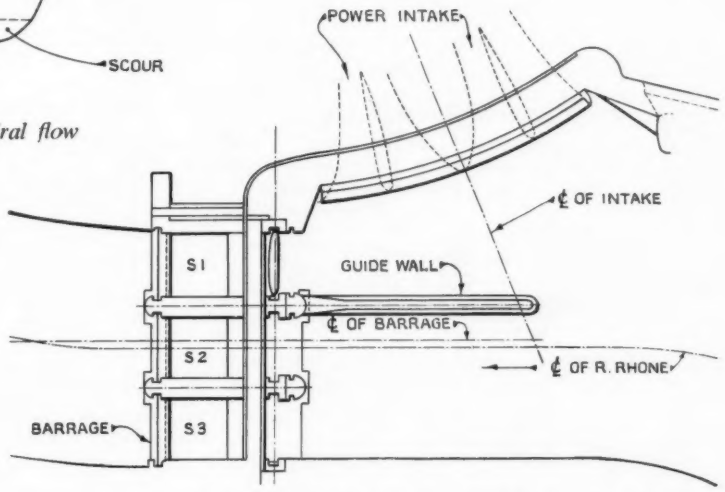

Fig. 3. Desilting system in use at the Morel and Lavey hydro-electric schemes in Switzerland

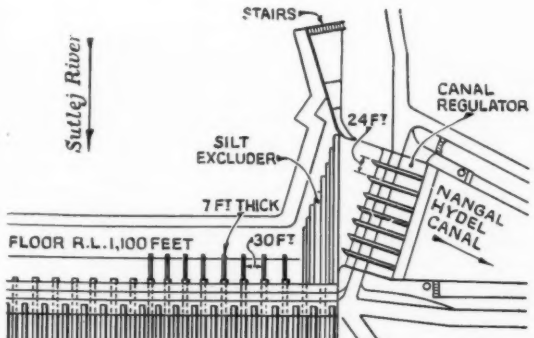

Fig. 2. General arrangement showing the intake for the power canal at Nangal dam

spans (because of the partial regain of velocity head) and thus a strong reverse flow is established at the bottom. This flow prevents any rolling material entering into the intake area. For medium discharges through the barrage only *S*2 is opened. In some other installations a scour valve may replace the gate in span *S*1. A low deflecting wall is sometimes provided around the intake to prevent the rolling material entering the intake.

In hill streams carrying heavy boulders the arrangement shown in Fig. 4 is sometimes adopted. Any superstructure built on this type of stream to raise the water level so as to keep the sill of the intake above the bed would be expensive. In such cases the bed of the stream is depressed in front of the intake. The silt deposited in the depressed bed is then sluiced out by opening the gate.

Another arrangement provided at the intake, especially at the forebay pond at the end of a long channel, is shown in Fig. 5. The sluicing of the silt deposit is affected by opening the valve intermittently. The disadvantage of this arrangement is that a portion of the deposit may not flow out. Also the deposited material may create a certain amount of tur-

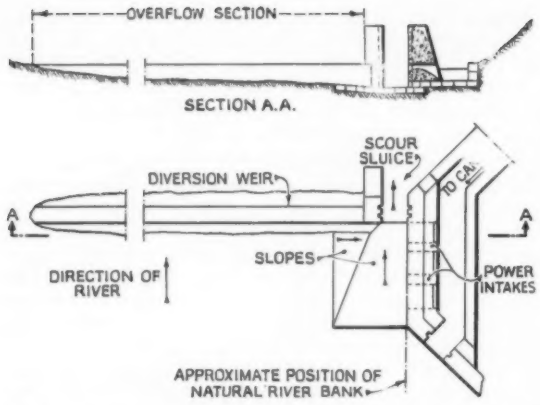

Fig. 4. System sometimes used for streams carrying heavy boulders

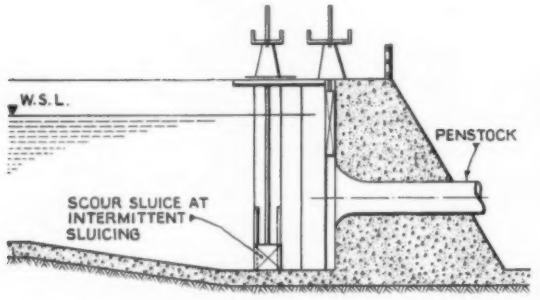

Fig. 5. A design frequently used in a forebay pond at the end of a long channel

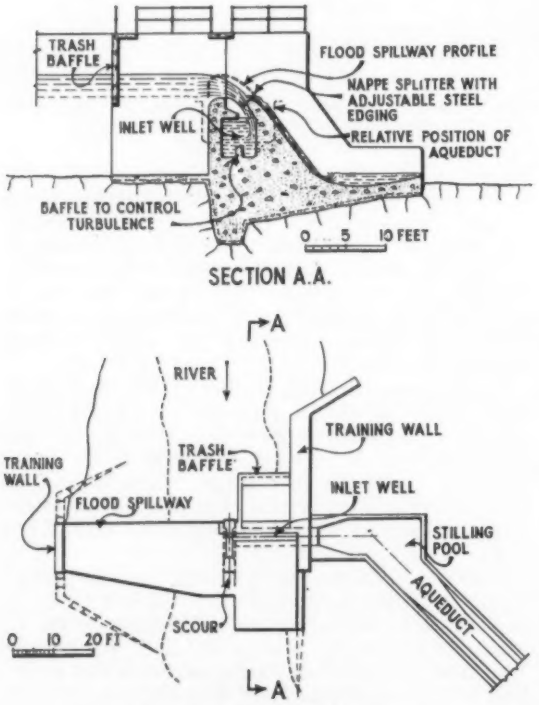

Fig. 6. Intake for major stream

bulence which raises it into suspension again so it escapes into the turbines.

Another interesting structure is that used on a system used for collecting water from intermediate catchments. Fig. 6 shows one of this type used at the Glen Shira project in Scotland[5].

Desilting Basins and Silt Excluders. In addition to the measures taken at the intake to prevent the entry of rolling silt, it is also necessary to arrest the silt in suspension from getting into the turbines. Desilting basins or silt traps are provided at suitable locations. Such desilting devices are based on the principle that a particle carried in the water flowing in a straight canal or tunnel tends to settle at the bottom.

Design of Desilting Basins

The conventional methods of determining the dimensions of desilting basins are empirical. They are usually based on the settling velocity of a sphere in a fluid:—

$$v=\sqrt{\frac{4}{3}\cdot\frac{g}{C_D}\cdot\frac{\rho_1-\rho}{\rho}\cdot D}$$

where v=settling velocity, C_D=drag coefficient, ρ_1=specific gravity of material, ρ=specific gravity of fluid, and D=diameter of particle.

The value of C_D varies with the Reynolds number R as given below:—

R	0·1	1	10	10^2	10^3	10^4	10^5	10^6
C_D	200	20	4	1	0·4	0·4	0·5	0·2

This velocity of settling is compounded with the velocity of flow in the basin and the settling distance determined. A factor of safety of 1·5 to 2 is then provided.

It is seen that the effect of turbulence on the settling of particles in the above method has been taken into account by adopting a suitable factor of safety. Attempts have been made to rationalise this by applying the theories of turbulent transport to the problem. Thomas R. Camp[6, 7] gives the removal ratio based on this theory as:—

$$r=1-8\left(\frac{vH}{2\epsilon}\right)^2\cdot e^{\frac{vH}{2\epsilon}}\sum_{\eta=1}^{\eta=\infty}\frac{a_h^2\,H\eta\;e^j}{\left[\left(\frac{vH}{2\epsilon}\right)^2+a_\eta^2+\frac{2vH}{2\epsilon}\right]\times\left[\left(\frac{vH}{2\epsilon}\right)^2+a_\eta^2\right]^2}$$

where v=settling velocity of particle, v_o="overflow rate" or discharge per unit area, ϵ=turbulent exchange coefficient, $j=-\left[\left(\frac{vH}{2\epsilon}\right)^2+(a_\eta)^2\right]\frac{1}{\frac{2vH}{2\epsilon}}\frac{v}{v_o}$, and H=depth of tank: a_1, a_2 and a_η are the successive real positive roots of the equation

$$2\cot a=\frac{a}{\frac{vH}{2\epsilon}}-\frac{\frac{vH}{2\epsilon}}{a}$$

$H_\eta=+1$ where a is in the first and second quadrant and -1 in the third and fourth. The roots of the equation can be obtained graphically by plotting the two sides of the equation and determining the points of intersection. A solution of this equation is given by Camp and is suggested for use by Hunter Rouse[8] for problems of desilting basins. In deriving

this equation, W. E. Dobbins' solution[6] of the differential equation for only one dimensional case of turbulent fluid was used. To adapt this solution to two-dimensional channel flow in a tank, Camp assumed:—

(1) Constant value of ϵ throughout the depth,
(2) Constant value of velocity v throughout the tank.

These two assumptions are evidently incompatible with each other because a constant value of ϵ assumes parabolic distribution of velocity. The value ϵ is obtained from velocity distribution curves. A parabolic velocity distribution which he suggests gives

$$\frac{vH}{2\epsilon} = \frac{122\,v}{V} \quad \text{where } V = \text{mean velocity in basin.}$$

The above result can only be approximate. V. Vanoni[9] found that there were differences between the concentrations of sediment obtained by his experiments on a channel and those obtained by the turbulent transport theory, though the agreement regarding the distribution of the sediment was good. He suggested that the difference was due to the variance of the value of the universal constant K in von Korman's universal Defect Law which was used in the determination of ϵ. This value was found to vary according to size and amount of sediment.

By using the theory of probability, Velikanov, Sokolov and others[10] have attempted to put the design on a more rational basis. A particle entering the basin at height h will probably settle in a distance given by

$$\frac{h}{v} = \frac{S}{V}$$

Where v = settling velocity, S = distance, and V = mean velocity of flow in the desilter if there is no turbulence. To take into account the effect of turbulence, the trajectory of a particle fluctuating up and down in the horizontal distance s with equal probability is considered. The vertical distance x from the horizontal path follows the normal distribution

$$y = \frac{1}{\sigma\sqrt{2\pi}} \cdot e^{\frac{-x^2}{2\sigma^2}}$$

where y = frequency of distribution and σ = root mean square of deviation.

With a particle of specific gravity greater than unity this has to be compounded with the settling of particle shown in Fig. 7. It is clear that the area of the distribution (shown hatched) below the floor in Fig. 7 represents the percentage of particles of that size

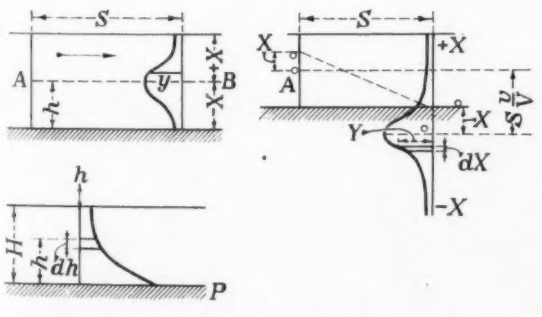

Fig. 7. The rate of settling

which have settled. The quantity of silt passing through element dx if there is no floor will be:—

$$dp = P_o\, ydx$$

where P_o = quantity of silt entering the basin. Therefore the quantity of silt settled is given by

$$P = P_o \int_{-\infty}^{x_o} ydx$$

$$\text{or } \frac{P}{P_o} = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{\frac{v}{V}s-h} e^{\frac{-x^2}{2\sigma^2}}\, dx$$

which represents the probability that any portion will settle in a distance s.

These expressions are simplified by introducing

$$t^2 = \frac{x^2}{2\sigma^2} \qquad \lambda = \left(\frac{sv}{V} - h\right) \div \sigma\sqrt{2}$$

The expression for λ contains σ the root mean square of deviations and can be determined only experimentally. Experiments were conducted with depth varying from 0·6 ft. to 4·30 ft., velocities varying from 0·3 ft. to 0·83 ft. per sec., silt rate from 0·25 g. per litre to 1·00 g. per litre and size of particles from 0·18 mm. to 0·57 mm. The relation between s and h was determined.

If $\psi = \frac{1}{\sigma\sqrt{2}} = f\,(s,\ h)$, then:—

$$\psi = \frac{2{\cdot}73}{\sqrt{sh}} \text{ or } \lambda = 2{\cdot}73\left(\frac{v}{V}\sqrt{\frac{s}{h}} - \sqrt{\frac{h}{s}}\right)$$

The fact that silt enters the basin at all levels should also be taken into account. Thus the silt that enters through a small element is

$$dp = pdh \qquad \text{where } p = f(h)$$

and the silt distribution along the depth is:—

$$dp = pdh\ \frac{1}{\sqrt{\pi}} \int_{-\infty}^{\lambda} e^{-t^2}\, dt$$

$$P = \int_{o}^{H} pdh\ \frac{1}{\sqrt{\pi}} \int_{-\infty}^{\lambda} e^{-t^2}\, dt$$

Since the entrance condition cannot be steady it is safe to assume uniform distribution or $p = \frac{P_o}{H}$

$$P = \frac{P_o}{H} \int_{o}^{H} dh\ \frac{1}{\sqrt{\pi}} \int_{-\infty}^{\lambda} e^{-t^2}\, dt$$

$$\text{or the removal ratio} = v = \frac{P}{P_o} = \frac{1}{H}\int_{o}^{H} dh \frac{1}{\sqrt{\pi}} \int_{-\infty}^{\lambda} e^{-t^2}\, dt$$

This formula is used to determine the dimensions of a desilting basin. The probability value for removing any particular size completely is fixed by economic considerations, having in mind the damage that would be done to the turbine against the cost of desilting works. It is usually fixed between 85 to 90%.

Defour desilting basins, based on the continuous-flow system, usually utilise 8 to 10% of the discharge. They have parabolic-shaped sides to obtain constant

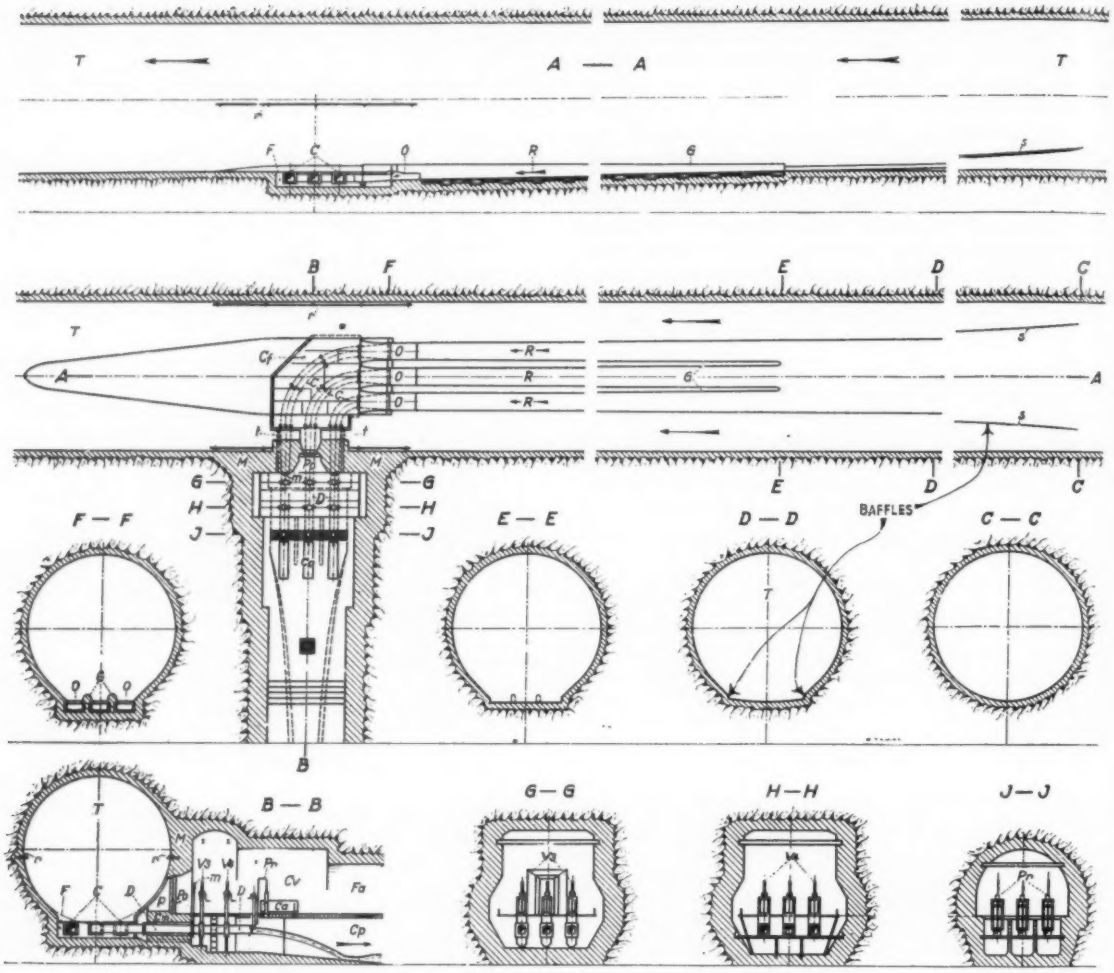

Fig. 8. Sand trap at Lavey

velocity. The construction of these shapes is rather expensive. On the other hand desilting basins where no attempt has been made to obtain constant velocities have been found to work very efficiently.

Another method of removing deposited silt is by an intermittent sluicing system. Here a certain volume for the silt reserve is provided in the basin and the sluicing is done periodically. Though there is considerable saving in water in this system, the defects mentioned in the arrangement, Fig. 5, are likely to occur.

The height of the basin should be sufficient to provide good sluicing velocities, but is restricted by the fact that greater heights would require longer basins. A working height of 12 to 15 ft. has been found to give satisfactory results. The velocity used in the basin may be about 0·75 ft. to 1·5 ft. per sec. for particle sizes of 0·25 to 0·5 mm. This may be increased to 2·5 ft. per sec. for bigger particles. Higher velocities will cause higher turbulence so that the settled material at the bottom may be put into motion and its removal will not follow theoretical considerations which assume that silt once settled does not re-enter into suspension.

Silt excluders are sometimes located in the water conduit system. A recent example is the excluder located in the tunnel near the surge tank at Lavey[11] (Fig. 8). The openings are not only designed to remove the rolling silt but also a portion of suspended silt for a certain depth above the floor of the tunnel. To accomplish this the passages are finished smooth and the velocity in the opening is maintained greater than the mean velocity in the tunnel. In the case of Lavey it was found that the velocity in the excluder would have to be about 1·5 times the mean velocity of flow in the tunnel. This velocity is regulated by the gates provided at the outlets. They are completely closed when the station demand is greater than the river discharge and the silt exclusion is done intermittently. The results obtained with this excluder are remarkable. On average about 350 tons of silt are removed every 24 hours. The size distribution of the silt removed is 56·29% $<$0·25 mm., 40·35% $<$0·57 mm. $>$0·25 mm., 3·29% $>$0·57 mm. $<$1·0 mm. and 0·07% $>$1·0. This shows that the excluder is capable of removing a good percentage of even the very fine material.

A similar arrangement has been adopted in the silt

ejector on the Nangal canal now in operation. Here a number of submerged tunnels, constructed of reinforced concrete, 6 ft. high and 8 ft. wide at the outlets, occupy the full width of the bed of the canal and are placed to discharge direct into the river. The depth at full-supply discharge (12,500 cusecs) downstream of the ejector is 20 ft. 6 in. The depth of water in front of the inlet of the tunnels is 28 ft. 6 in. The canal upstream has been designed with a full supply depth of 23 ft. for a discharge of 14,500 cusecs, this including 2,000 cusecs for ejection of silt through the excluder. The extra 5 ft. 6 in. of depth from the bed of the canal to the sill of the tunnel has been constructed in the form of a gradual slope. Hydraulic models prepared for the study of this ejector gave good results.

Conclusions

In order to prevent damage to the turbine caused by erosion and so ensure efficient operation, it is essential to eliminate the entry of silt of such sizes that are injurious to the runner and discharge ring. The methods which have been evolved by theoretical and hydraulic model studies are found to be satisfactory.

Preliminary studies of silt analysis are absolutely necessary to determine the distribution of particle size and the mineralogical composition of the silt contained in the water diverted for power generation. These studies are particularly important in the case of run-of-river projects constructed on streams which flow on a movable bed and carry in suspension a large quantity of coarse silt. Added to the silt-suspension studies, hydraulic model studies to evolve an effective method of silt exclusion greatly facilitate an economical solution to these problems. The time and money spent on these studies are fully justified in the resulting gain in efficiency and trouble-free operation of hydraulic turbines.

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Type M.44 Douglas Dumper

Douglas Equipment Limited have previously produced four-wheel drive chassis of varied designs on which various types of equipment, including tipping bodies, have been mounted. It is on these basic components that the Douglas Four-wheel Drive Dumper has been designed.

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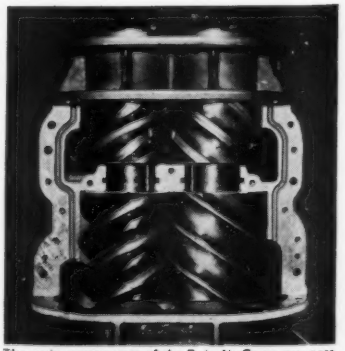

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of 9 sec. The same chassis has been sold to Sir Robert McAlpine & Sons Limited and fitted with a Dumpcrete body, and is being operated on the Bradwell atomic power station site. The M.44 6–8 cu. yard dumper is available for demonstration, and is offered at a very competitive price to both home and overseas markets. The short wheelbase and four-wheel drive make it highly manoeuvrable for operation under difficult conditions. The engine is a high-speed Rootes Lister two-stroke diesel, which, together with eight-speed gearbox, gives a tractive effort of approximately 16,000 lb. and provides a wide range of performance. Alternative diesel or petrol engines are available on request.

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Eimco Model 622 RockerShovel

The model 622 RockerShovel is the latest addition to the range of heavy-duty loading machines developed by Eimco (Great Britain) Limited, Team Valley, Gateshead, Co. Durham. Seen in the accompanying illustration, the Eimco 622 is crawler mounted and built to withstand arduous working conditions. It is available with either compressed air or flameproof electric drive.

For pneumatic operation, two five-cylinder radial air motors each developing 12 h.p. provide the traction, one driving each track. A third identical motor operates the bucket and rocker arm assembly. Alternatively, the machine can be supplied for electrical operation for supplies ranging between 380 V and 660 V a.c. with flameproof Buxton certificated motors and control gear suitable for Class I and Class II atmospheres.

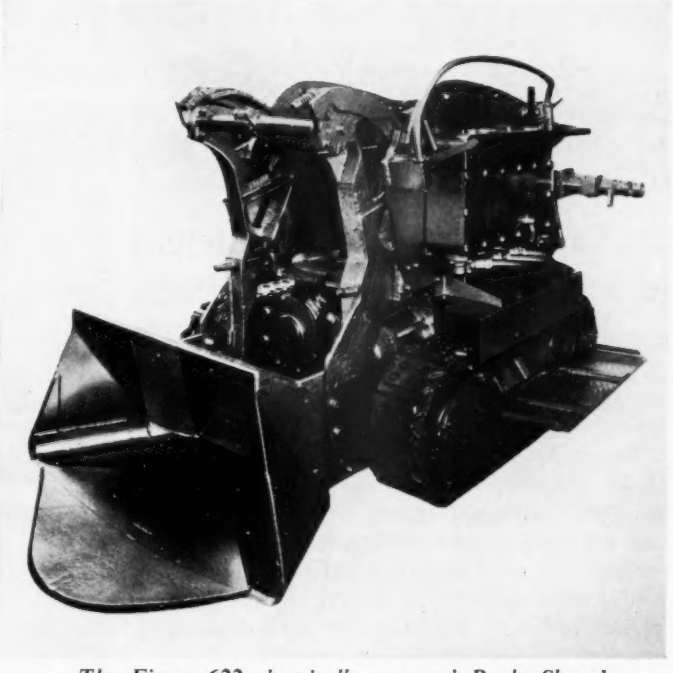
In confined spaces in tunnels, and vertical shafts, the simple overhead rocker arm action is said to give five to six full buckets a minute, each of half a cu. yard at a time. The independently driven and controlled tracks allow rapid manoeuvrability, and spin turns can be made, permitting the machine to turn virtually within its own length.

The half cu. yard bucket covers the full width of the tracks and, by a simple adjustment, four separate digging positions are provided, varying from above track level to 4 in. below track level.

A heavy-duty, open-type track is employed, flat track shoes being supplied on the standard machine; however, either a full or part grouser track can be fitted when necessary. The sprung front idler absorbs operating shocks and allows rapid, simple track tensioning.

Electrically-powered versions of the model 622 RockerShovel are fitted with a hand-controlled track brake which enables the operator to place the machine with ease and accuracy when coming up to discharge the bucket.

On both models the overall width is 5 ft. 9½ in., the headroom requirement varying from 8 ft. 10 in. to 9 ft. 1 in. and the discharge height from 5 ft. 2½ in. to 5 ft. 11½ in. Alternative discharge heights are available to meet special requirements. The overall length of the air machine with the bucket down is 8 ft. 9 in., while that of the electric machine is 9 ft. Travelling speed is 2¼ m.p.h. The weights of the two versions fitted with flat track shoes are given as 9,000 lb. for compressed-air powered models and 10,750 lb. for the electric.



The Eimco 622 electrically powered RockerShovel

Compressed air consumption varies according to load from 350 to 400 cu. ft. per min., with an air pressure range of 60 to 120 lb. per sq. in. The electric power supply range (a.c. only) is 380 to 660 V.

Tracto-Loaders. A coloured brochure received from Mackay Industrial Equipment Limited gives particulars and illustrations of the Allis-Chalmers range of earth-moving equipment.

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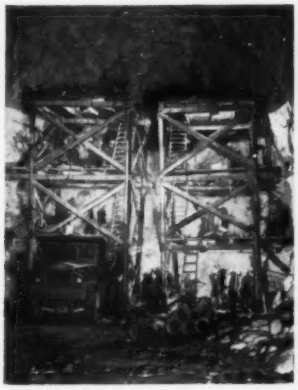

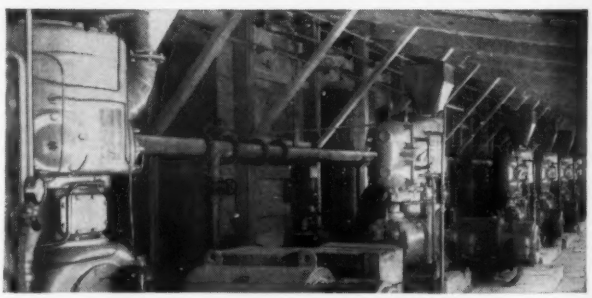

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DIS 49

Book Review

Hydraulic Turbines. Their Design and Equipment. By Prof. Miroslav Nechleba, Dr. Tech. M.E., and translated by C. Meyer, C.H.E., and A. G. Evans. Published by Artia–Prague, and Constable & Co., Ltd., 10–12, Orange Street, London, W.C.2. Format: 8 in. x 6 in.; pp. 636, 545 ff. Price £2 5s. 0d.

The author of this treatise is Professor of Hydraulic Machinery at the Slovak Technical University in Bratislava, and was awarded a State Laureateship for theoretical and practical work on hydraulic turbine construction and control. Although he is best known in Western Europe for his original work on high-head kaplans, this astonishingly inexpensive book, in English, and references to others in the bibliography, indicate the breadth of his activities in the hydraulic turbine field.

When it is realised that the last English text books on Hydraulic Turbines were written by Gelpke & Van Cleve (1911) and Daugherty (1920) it will be appreciated that a new one has long been needed. This is also confirmed by the author's references to classical French and German works such as Tenot (1935), Thomann (1935) and Camerer (1924) on turbines and Fabritz (1940) on governing, which are now out of print. Apart from its technical value, the book gives an interesting glimpse of eastern European designs and the extent to which western techniques are being utilised.

This treatise on hydraulic turbine, utilises elementary mathematical methods to give an answer to all but the most specialised problems. It is unusual in that the author also devotes considerable space to the important phase of structural design and supplements theoretical approaches with numerical examples which show that he appreciates the needs of the designing engineer and draftsman.

The book is logically arranged in four parts, a brief survey of which may be of value.

Part I (122 pages) is an introduction to hydraulic theory and covers such basic questions as turbine classification, forces and movements in stationary/revolving ducts and output control illustrating them with diagrams and drawings of francis, propeller, kaplan and pelton turbines. The sections dealing with hydraulic similarity lead up to the conception of specific velocities and specific speed and their extension to efficiency variations in terms of machine dimensions and head and subsequently to topographical diagrams based on unit values for generalising turbine performance. This is an easily understood treatment which will be welcome to those encountering allied problems for the first time. They include the derivation of Moody and Camerer step-up formulae and a reference to Hutton's work on kaplans. Friction, shock and discharge losses are not neglected and the fundamental treatment concludes with an algebraic and geometrical analysis of runner inlet/outlet triangles culminating in Braun's diagrams and their application to francis turbines and kaplans with fixed and adjustable guide vanes.

Part II (300 pages) applies the material in Part I to hydraulic and mechanical design. Francis turbines, to which 188 pages are devoted, set the pattern for study of propeller and kaplan types, since spiral casing, draft tube, turbine covers and guide apparatus are common to all reaction turbines.

Francis runner design for low and high specific speeds employs the one, or two, dimensional approach using meridional flow fields, velocity triangles, constant circulation and conformal representation to derive runner inlet/outlet diameters, inlet/outlet edge positions, blade shape, cross section and thicknesses and duct shape including reference to the influence of oblique guide vane discharge and the draft tube.

This lucid analysis is correlated into a blade layout procedure illustrated by numerical examples. This is coupled with an original theoretical treatment of blade strength where the empirical Camerer and Thomann formulae are replaced by one based on applied forces and stresses.

Hydraulic thrust components are derived mathematically and associated with less rigorous empirical formulae. Labyrinth leakage losses, shaft loads and critical speeds are usefully outlined.

The number of guide vanes and their aerofoil profiles is based on minimum loss and constant contraction from which resultant open and closed position moments are calculated. Finally a theoretical approach is made to spiral casing shape in conjunction with the stay vanes and their relation to the guide vanes. From a practical viewpoint this section, and also the one dealing with draft tubes, are, perhaps, a little weak in that they lack guidance on normal dimensions. The spiral casing including stay vanes, inner and outer covers, with their shaft glands are examined for hydraulic loading and consequent stressing and dimensioning in vulnerable places. Draft tube notes include references to steel and concrete linings and air valve dimensions in terms of guide-vane closing time.

A glimpse of composite, integral and split casting runner construction is given, as well as a summary of cavitation theory. It would appear that cavitation research is proceeding much on the same lines as in western Europe and elsewhere in terms of mechanical fatigue and intercrystalline corrosion. There is also a note on cavitation similitude in model and prototype.

The section on controlling mechanism is admirably handled in that it defines the guide-vane and spindle loads and deals with the mechanical design of the regulating ring in a novel mathematical manner.

The theoretical treatment of propeller and kaplan turbine runner design applies, in readily understandable terms, the properties of NACA Report 460 Aerofoils to lattice system conditions. This merits study and leads us to consideration of the turbine space shape, defined by runner hub, chamber, cover and guide vanes, meridional flow field and inter-related flow surface velocity diagrams which ultimately determine the number of blades and hub ratio in terms of head and specific speed. An examination of blade flow stability, and blade and gap cavitation based on Shalnev's work, is included.

Mechanical problems related to blade and shaft stresses, hydraulic thrust at normal and runaway speeds and sudden blade closure, hub, bearing, shaft and inner cover design, complete this concise analysis with comprehensive but expert facility.

Some sixty pages on pelton turbines were con-

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tributed by Ing. M. Druckmuller who gives a competent treatment of specific speed, jet and wheel diameter and bucket calculations, extending them to jet and centrifugal forces in relation to lug and bolt dimensions. Theoretical and practical considerations concerned with needle, nozzle and deflector dimensions, include determination of the active forces concerned with the hydraulic and/or spring-loaded operating mechanism design.

Part III on turbine equipment devotes one informative section to turbine governors embracing the principles of servomotors, differential control valves, restoring mechanisms and speed sensitive devices, and some comments on electrical drives. An interesting item is the theory and description of the CKD—Blansko governor which claims to embody an ideal massless hydraulic speed responsive element. It also offers some data on pressure regulators, oil pumps, unloading valves, pressure receivers and air/oil volume control.

The short section on speed regulation, stability and matching of machine and governor characteristics to network requirements, including the effect of pressure relief valves, is an excellent introduction to more advanced works.

A chapter by Ing. J. Urban dealing with thrust and guide bearings is a valuable addition and covers a comprehensive outline of hydro-dynamic lubrication theory correlated with illustrations of actual designs. Segment materials, collar design, oil characteristics, generator brackets and questions such as oil/water cooling arrangements, are briefly but competently treated.

Turbine and governor designers have a considerable interest in pipeline and surge chamber questions. The section on these topics will attract their attention, because the precise determination of pipe dimensions, stresses, economic span and anchor-block stability, should make immediate reference to advanced tests unnecessary for preliminary work.

The fundamental approach to waterhammer theory, the Schnyder Bergeron diagrams and complex surge chambers using Schoklitsch's graphical method are worthy introductions to the techniques employed by specialists.

This section concludes with short but interesting paragraphs dealing with quick closing devices, intake gates, butterfly and rotary valves. These are special and important devices, but enough is presented to suggest that something more comprehensive might be added in future editions; too little is known or published about their design and performance.

Part IV deals, in five pages, with the main points which demand attention in commissioning a unit for the first time. The current meter method of flow measurement is briefly described, while the Gibson and thermodynamic methods are mentioned in passing.

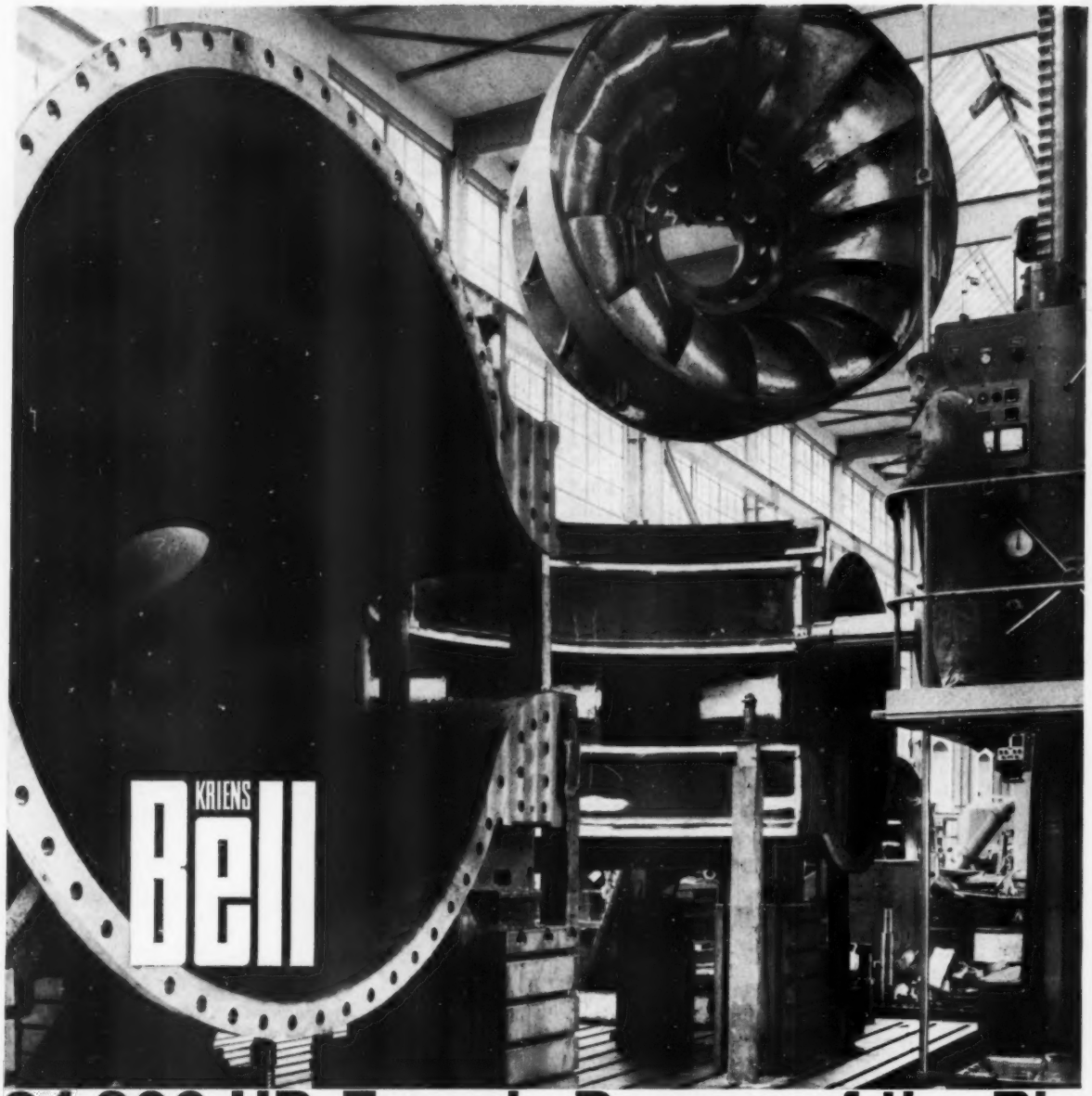
There are many references to Russian and Czechoslovak literature which reveal the need for an adequate translation service in this field. If Nechleba's book on "Theory of Indirect Speed Control (1952)" fulfils the standard of its companion on "Hydraulic Turbines," it should be translated without delay.

Although the book is printed in Czechoslovakia in clear type with excellent diagrams and drawings and is adequately indexed, the high standard of translation suffers a little from unusual and inconsistent terminology, numerical, spelling and idiomatic errors, the correction of which would ensure the undistracted understanding of an otherwise very readable and valuable work.

Abstracts from the World Technical Press

The Inn-Spöl Development

The importance attached to this project in Switzerland is best reflected in the wide publicity it has been given not only in all the technical journals but also in the daily papers. Once more the trouble lies in the superabundance of beauty spots in the country, and, following a rule which has but very few exceptions, this project has brought about a sharp conflict between the local fanatics of the "hands-off-our-landscape" policy and the supporters of the widest possible harnessing of hydro-power resources in the general interest. As in many previous instances, the final decision was left to a national referendum, and since the project affects Italian interests (the Spöl crossing the Italo-Swiss border at the upstream tip of the Lesigno reservoir) and has been sanctioned in May 1957 by an agreement between the two countries, the question submitted to the referendum was simply the ratification of the so-called Spöl Treaty. The opposition was most pronounced in the Grisons, the works of the scheme being sited to a considerable extent within the boundaries of the Engadine National Park. The referendum, held on December 7, 1958, decided heavily in favour of ratification by 500,933 ayes to 165,550 noes, 55% of the registered voters abstaining. It appears now certain that the 1958 Inn-Spöl project of the Engadiner Kraftwerke A.G. will soon reach the construction stage; this "compromise project," as it is called, was actually approved in 1957 but subsequently amended by substituting a 6·5 million cu. m. balancing reservoir at Ova Spin for the 28 million cu. m. Praspöl storage reservoir which would have cut the National Park in two. The backbone of the scheme is the 180 million cu. m. Livigno reservoir, with a storage level 1,808 m. above sea level, situated entirely on Italian territory close to the Swiss border, and from which the Spöl flows in a north-west direction until it joins the Inn near Zernez. The head storage of the development, however, is the 55 million cu. m. Chamuera reservoir, 2,093 m. above sea level, from which a pressure tunnel 7·6 km. long feeds the 37,000 kW S-chanf power house, the tailwater of which is collected by a 12·8 km. free-flow tunnel connecting at its downstream end with the 7 km. pressure tunnel from the Livigno reservoir. From the 43,000 kW power station and 28,000 kW pumping plant at Ova Spin, a 21·6 km. long pressure tunnel 5·2 m. in diameter leads to the 240,000 kW plant at Pradella, the tailwater of which feeds the 56,000 kW



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downstream plant at Martina through a 14·1 km. pressure tunnel. The aggregate installed capacity of the project is therefore 376,000 kW, giving a yearly production of 1,310 million kWh split about equally between the winter and summer periods, the share of the Pradella plant alone amounting to 980 million kWh. Other characteristics: aggregate reservoir capacity 241·5 million cu. m.; installed discharge per second and maximum gross head: S-chanf 11·5 cu. m./442 m., Ova Spin 33 cu. m./178 m., Pradella 66 cu. m./487 m., Martina 70 cu. m./113 m.; Ova Spin pumping plant 12 cu. m. per sec.

NOTE.—The above abstract has been drawn up from the following journals: *Schweizerische Bauzeitung*, Vol. 76, No. 46, November 15, 1958, p. 685, 10 pp., 4 ff., and No. 52, December 27, 1958, pp. 789/790; *Tecnica*, Vol. 7, No. 24, November 21, 1958, p. 1365, 8 pp., 11 ff.; *Wasser- und Energiewirtschaft*, Vol. 50, No. 11, November 1958, p. 303, 29 pp., 20 ff., 9 maps, a special issue devoted to the Inn-Spöl scheme, headed by an article in which Dipl. Ing. G. A. Töndury discusses the question in all its aspects.

Lika-Gacka Development

The installation of this Yugoslav development is described in broad outline. It is a high-head scheme, drawing its water from a seasonal reservoir provided upstream of the diversion canal, the tailrace discharging direct into the Adriatic Sea. The underground power house at Senj, close to the shore, south-west of Zagreb, will be equipped with three generating sets consisting each of a vertical-shaft Francis turbine connected to an 80 MVA synchronous alternator. Three-phase transformers will feed the transmission system at 220 kV. (K. Begović, *Energija*, Zagreb, Vol. 7, No. 7/8, July-August 1958, p. 197, 10 pp., 15 ff.)

The Frayle Reservoir and Dams

This 200 million cu. m. reservoir is situated due west of Lake Titicaca at an altitude of 13,410 ft. above sea level, i.e. almost 900 ft. higher than the lake. It has been created by damming Rio Blanco, a left-bank tributary of Rio Chili which flows on Peruvian territory roughly parallel to the Bolivian and Chilean borders. The main dam, a double-curvature or dome-type arched concrete structure, which closes a deep canyon between practically vertical walls, has the following main characteristics: 74 m. maximum height, 6·2 m. thick at the base and 1·55 m. at the crest, 11,000 cu. m. in volume, foundations excluded. At the top, where the canyon widens, the dam is flanked on either side by a gravity wing. The radius of the planimetric curvature at the crest is 25 m., the chord of the arc being 40 m., and the developed length of the crest, wings included, is 90 m. The central section of the dam is designed as an overflow weir, the four 30 m. openings of which can pass a discharge of 135 cu. m. per sec. For irrigation purposes, a tunnel 2·9 m. in diameter driven in the left bank of the river ensures a flow of 47 cu. m. per sec., which in cases of emergency can be increased to 102 cu. m. In addition, an emptying conduit controlled by a 750 mm. diameter valve is provided in the right flank of the canyon at the level of the river bed, so that the bottom of the reservoir can be laid bare for inspection, the conduit also acting, when required, as an additional flood-discharge outlet. The foundations of the dam required 7,200 cu. m. of excavation work. In order to raise the maximum level of the reservoir to 4,010 m., so as to reach a full capacity of 200 million cu. m., an indentation on the right bank of the river, about 1 km. upstream from the main dam, was closed by a side dam of concrete blocks. This slightly curved side dam has a maximum height of 23 m., a volume of 28,000 cu. m. and a developed crest length of 266 m. The total area covered by the reservoir measures 14 sq. km. The main dam, sited far away from any inhabitated place or communication centre, called for the laying of a 28 km. access road, reaching 4,800 m. above sea level at its highest point, and leading to the left bank of the canyon. Since the side dam is on the other bank, the cableway set up at the preliminary stage of construction work had to be replaced first by a footbridge and subsequently by a bridge 25 m. in span, designed for loads of up to 25 tons. Work on the Frayle reservoir and dams began with the opening of the access road in May 1956, and the volume of irrigation water available reached 20 million cu. m. in the first year of operation. The project was worked out by the hydroelectric construction department of the Italian Edison Group, who also acted as consulting engineers for the dams. Construction work was carried out by Società Panedile Peruana. (Ecc., *Quaderni di Studi e Notizie*, Vol. 14, November 16, 1958, p. 779, 5 pp., 5 ff.)

The Wadrinau Run-of-River Plant

A summary description is given of the Wadrinau hydro plant of the Metz Corporation, on the Mosel River. Seen from outside, the plant presents the classical layout of a power block and a spillway block built in one line from one bank of the river to the other. The generating equipment, however, consists of horizontal pit-type sets. The spillway block, on the left bank of the river, consists of two 45 m. openings with sector gates 6 m. high and stiffened by steel cylinders 2·5 m. in diameter. These gates are operated by automatic winches housed in fairly low cubicles, which from a distance are practically the only visible part of the superstructure. The two spillway openings can deal with the highest flood discharge occurring on the Mosel River, viz. 2,200 cu. m. per sec. In case of such a flood, the gates are lowered into a groove provided in the apron of each opening so as to enable the water to flow unimpeded throughout the full cross section of the opening. A gallery running along the whole length of the foundation block of the spillway provides access to the gate winches, and also acts as a drainage gallery. The normal level of the pool is 165·28 m. and the minimum downstream level 160·68 m., so that a normal operating head of 4 m. is available. The power-house block, partly embedded in the right bank of the river, is divided into four identical sections 10·5 m. wide, each equipped with a horizontal generating set consisting of a tubular-type turbine and an alternator erected in an open pit. The four sections of the power house are erected on a common foundation block and apron, their top parts being connected together by the rails of the travelling service crane and of the trashrack, as well as by a footbridge above the draft tubes. The generator pits

are oval in cross section, and their outer walls form the supply channels to the turbines. The downstream chambers have been designed to act as surge tanks. The control room of the plant is divided into two compartments, one housing the drainage pumps as well as the pump evacuating the debris caught by the trashrack into a flush channel, the other containing all the 17 kV equipment as well as the set of relays ensuring the remote control and automatic working of the plant. The turbines are of the Kaplan type with adjustable blades and conical distributor; they are fixed on the lower part of the distributor embedded in the concrete of each generator pit. Each 1,690 h.p. turbine, designed for a discharge of 35 cu. m. per sec. from a 4 m. head, drives its 1,500 kW, 1,000 V alternator by means of a speed-increasing gear. Each alternator is coupled direct to a transformer standing higher up in the pit on a platform, and stepping up from 1,000 to 17,500 V, which is the voltage of the Metz Corporation system operating independently of EDF. Efficiency is 91·5% for the turbine alone under a 4 m. head, and 84·3% for each complete set, speed-increasing gear and transformer included. Yearly production is assessed at 26 million kWh. The economical layout of the whole plant is practically the same as suggested for tidal power plants, and the Wadrinau plant is expected to add a useful contribution to the series of tests in progress at Argentat and other plants of the same type. (*Le Génie Civil*, Vol. 135, No. 20, October 15, 1958, p. 423, 3 pp., 4 ff.)

NOTE.—The above-mentioned journal also contains a general survey of Turkey's hydro-electric developments. (pp. 425/426, 1 map.)

Adapting Existing Hydro Plants to Automation

The authors discuss in this article the advisability of transforming existing manually controlled hydro plants into automatic plants. The fundamental prerequisite of automation is in every instance the extent of profitability realisable. In this respect the position between industrial or manufacturing plants and energy-producing undertakings differs in this, that automation of the former aims at simultaneously increasing production and reducing operating and manufacturing costs, whereas in the latter its goal is to reduce operating and supervising personnel, that is to say, labour, without any increase in output. The actual instance taken into consideration as an example is a small autonomous run-of-river plant, the incorporation of which into a co-ordinated chain of plants on the same river made its adaptation to automatic working imperative. The generating set of the plant consisted of a 110 h.p. vertical Francis turbine passing 4 cu. m. per sec., coupled by gearing to a 120 kVA horizontal generator, producing in a normal year about 580,000 kWh. The main requirement of automation lies in a fully reliable quick closure of the turbine, and this was achieved by transforming the existing slidegate into an automatic dropgate, discarding the turbine governor, and adding to the equipment a centrifugal switch to stop the turbine in the case of a failure in the parallel working of the system, and all the necessary control, safety and signalling devices. In addition, it was necessary to install a 125 kVA transformer, to reconstruct the switchgear, and to connect the plant by means of a 900 m., 25 kV overhead line to the neighbouring downstream plant of the chain. Civil-engineering work was practically restricted to improvements in the intake structure. In the authors' opinion, automation can be carried out as described at relatively low cost, provided the small plant involved is situated at a fairly short distance from a normally attended major power plant. Above all, no adaptation of such a plant to automatic working should be carried out without a previous thorough investigation of its profitability. Small or medium plants designed for a substantially higher output than the 120 kVA instanced must be considered in the same light as large plants, since the problems they present are much more complex, especially as to governing. In fact, the automation of such plants might easily prove unprofitable, owing to the additional equipment they require and the amount of expenditure they involve. It is therefore advisable, when dealing with existing installations, to apply automation to smaller plants only. (F. & E. Petter, *Oesterreichische Zeitschrift für Elektrizitätswirtschaft*, Vol. 11, No. 12, December 1958, p. 502, 5 pp., 6 ff.)

The Madras Grid

This article describes the progress achieved in power generation in the Madras State since 1908, when there were no other installations than a few tiny plants run on water power by tea estates, and the small hydro plant at Kateri in the Nilgiris. The first public electricity supply was initiated by the Madras Electricity Supply Corporation under licence in Madras City itself in 1909, but real progress began 16 years later when the Government of Madras decided to develop itself the hydro-electric power resources available on its territory. At the present moment, the Madras grid comprises four hydro-electric stations aggregating 174,200 kW in installed capacity, plus two thermal plants totalling 112,500 kW. Transmission facilities consist of 10,500 miles of high-voltage lines, varying in voltage between 11,000 and 110,000 V, 13,900 miles of low-voltage lines, 56 EHV and 61 HV substations, and 6,642 distribution stations. Since the demand is expected to reach 549,000 kW in 1961–62, new power sources have to be tapped. Investigations concerning two of the most important schemes have just been completed: the Mettur Tunnel scheme, which will utilise the irrigation discharge of the Mettur dam and give 200,000 kW seasonal power for seven months of the year, and the Kumbar-Amaravathi hydro-electric scheme in the Palni Hills which will add 20,000 kW to the grid. It is further expected that the Government of India would make 200,000 kW available to the Madras grid from the 250,000 kW thermal plant contemplated in connection with the integrated Neyveli lignite scheme. (V. P. Appadurai, *Indian Journal of Power and River Valley Development*, Vol. VIII, No. 9, September 1958, p. 3, 5 pp., 1 map.)

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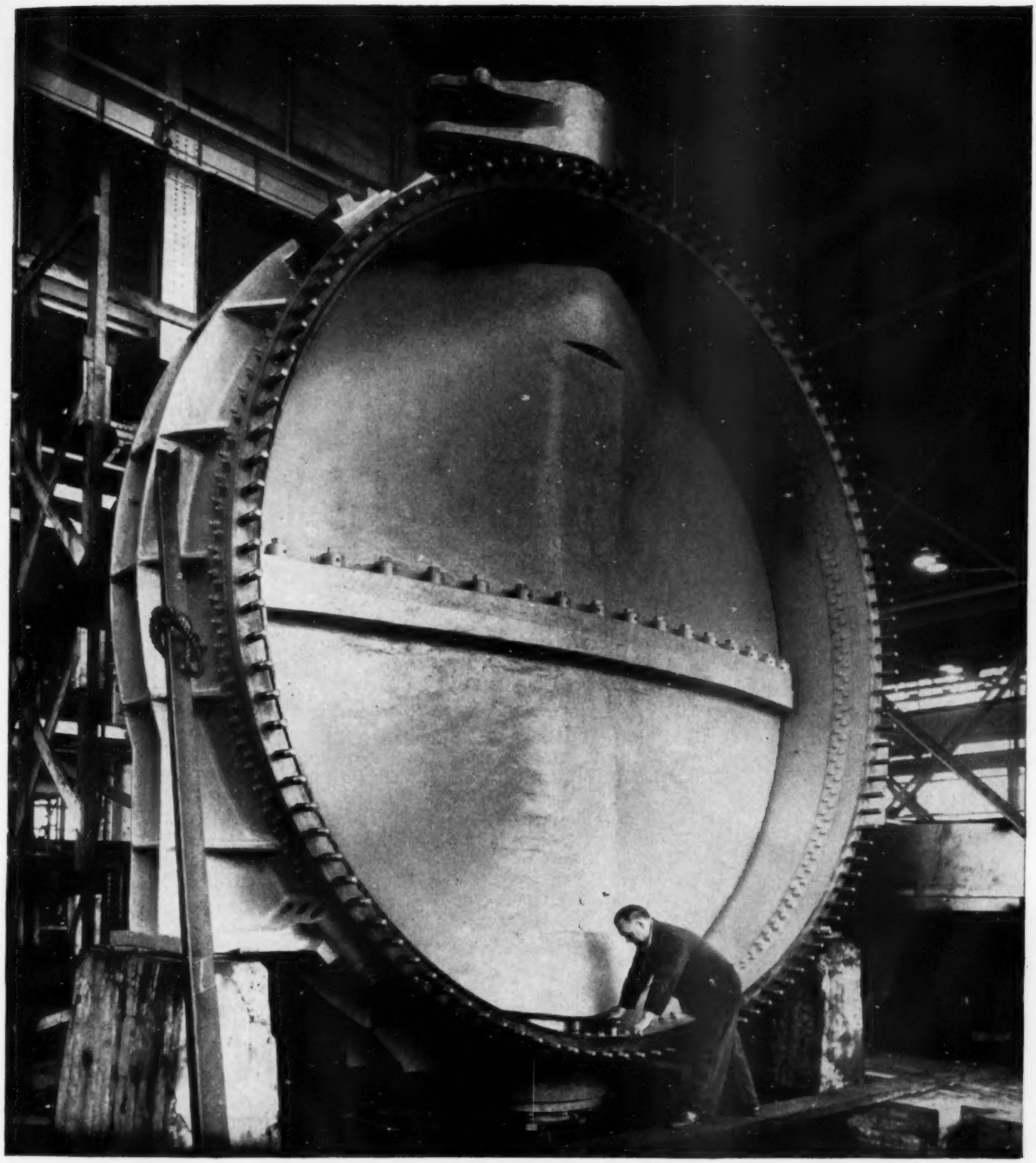

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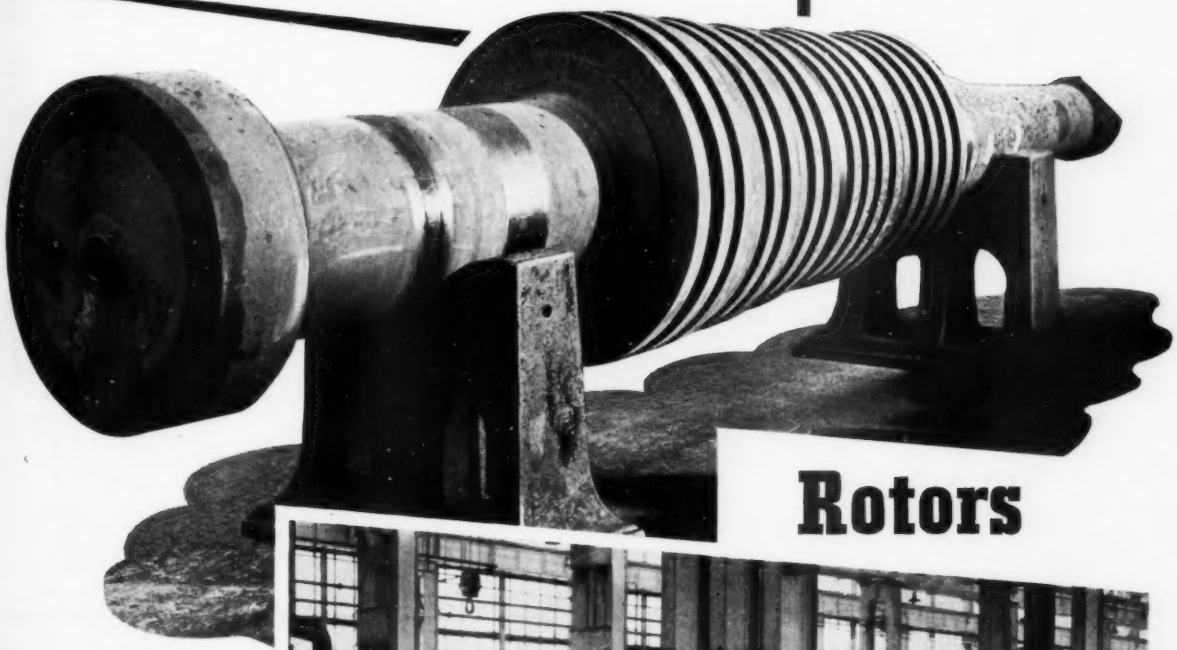

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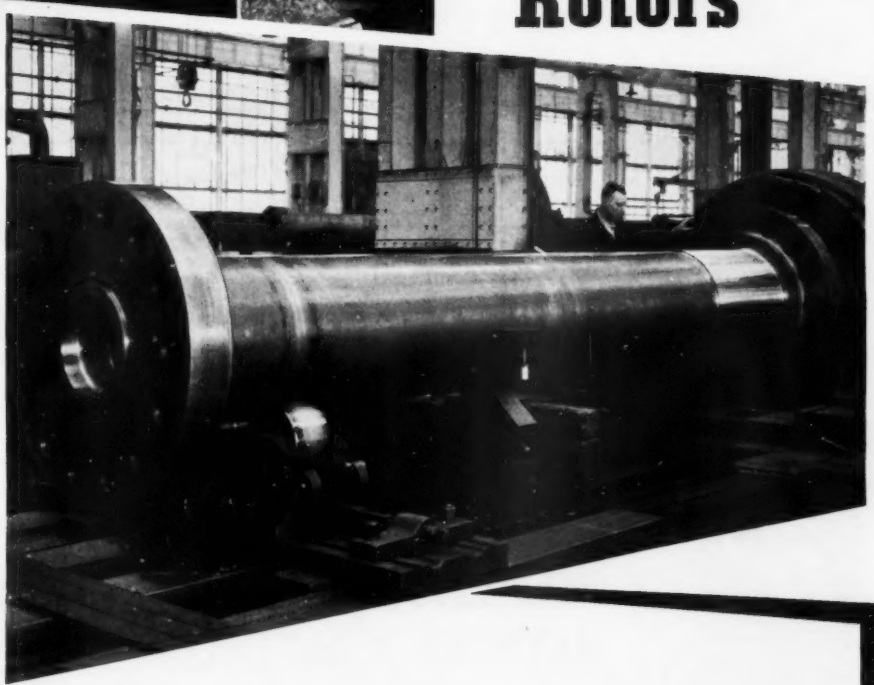

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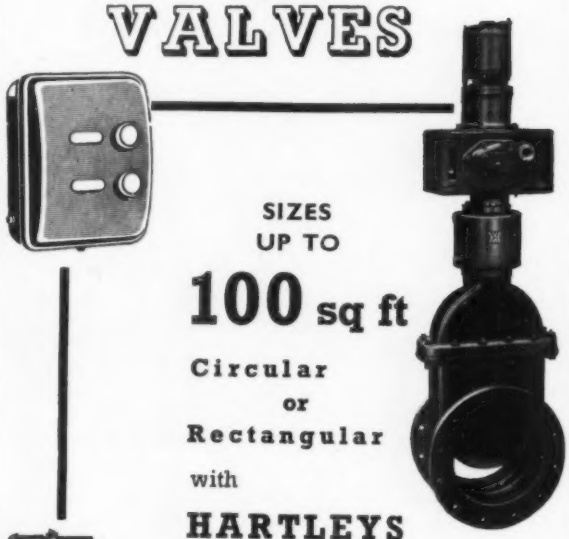

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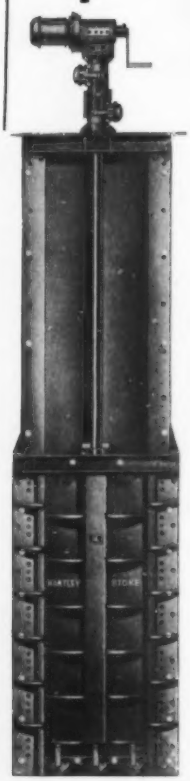

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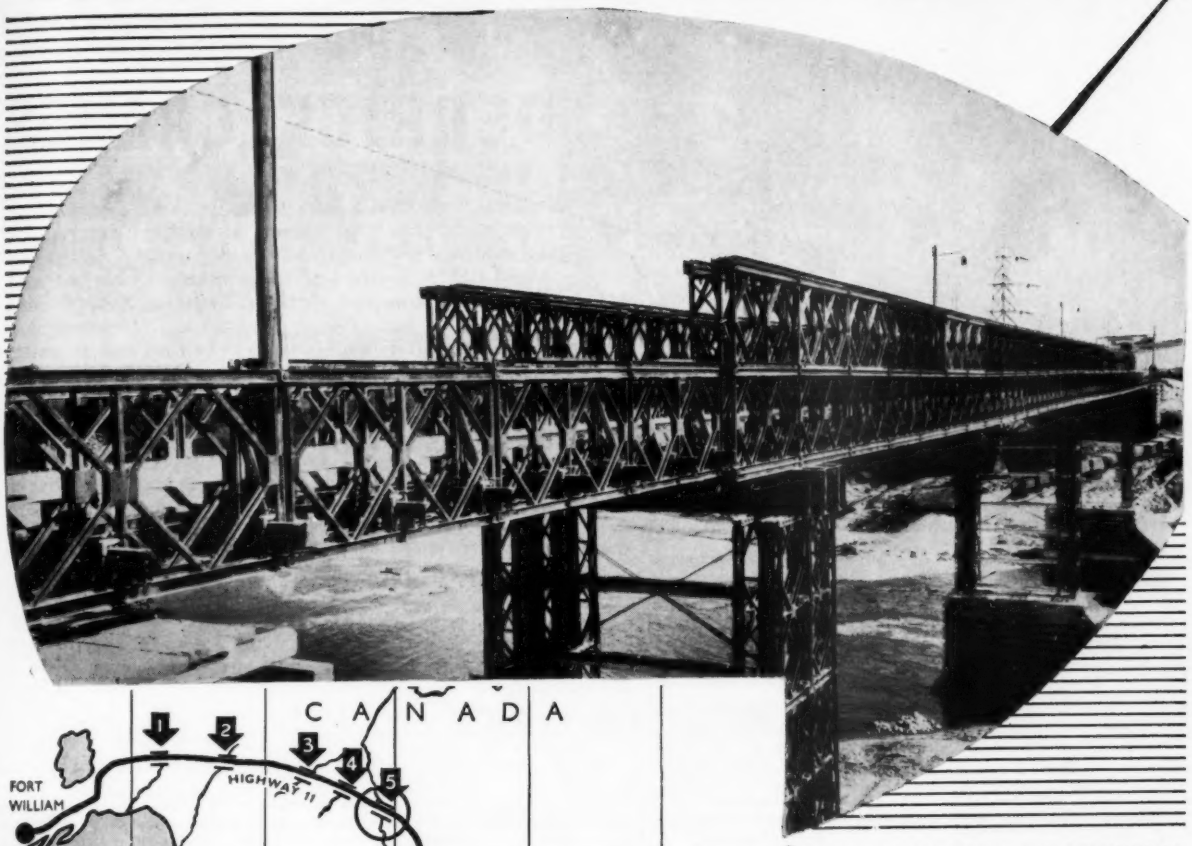

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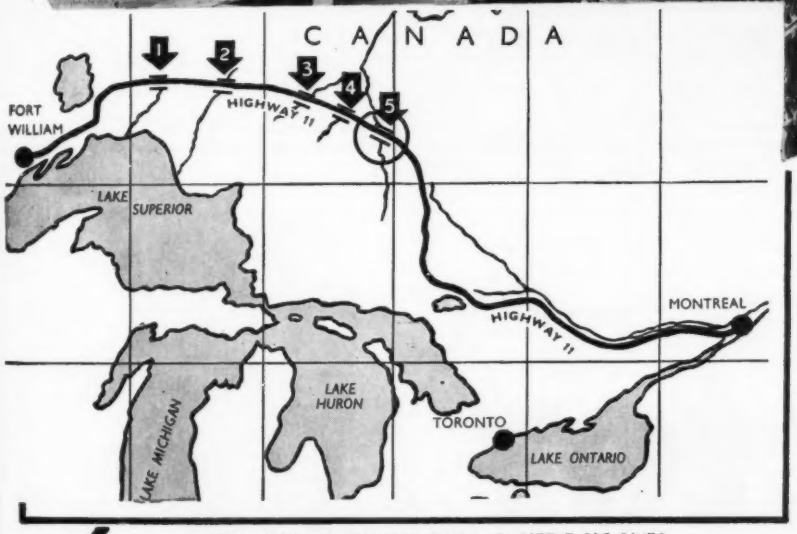

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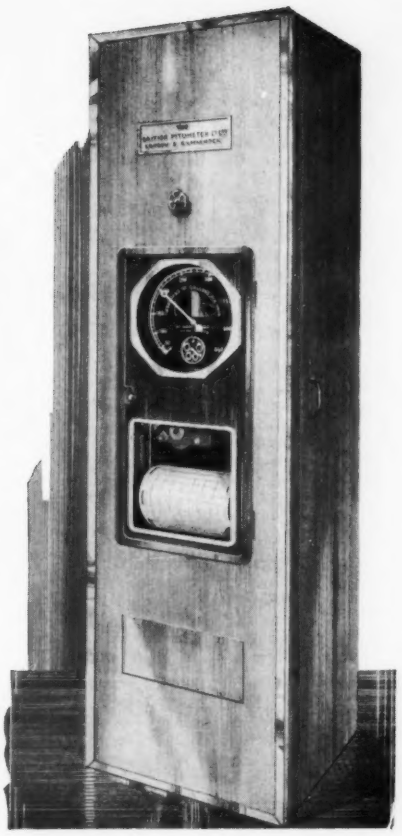

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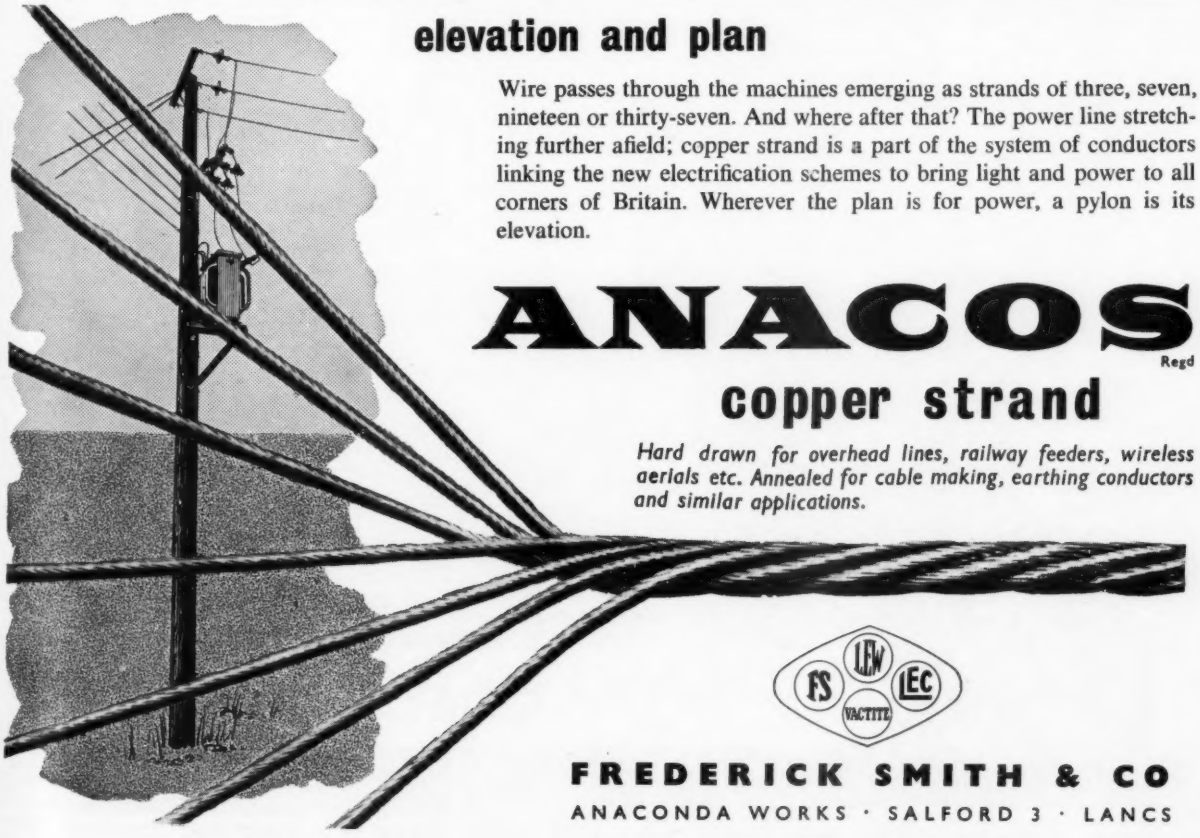

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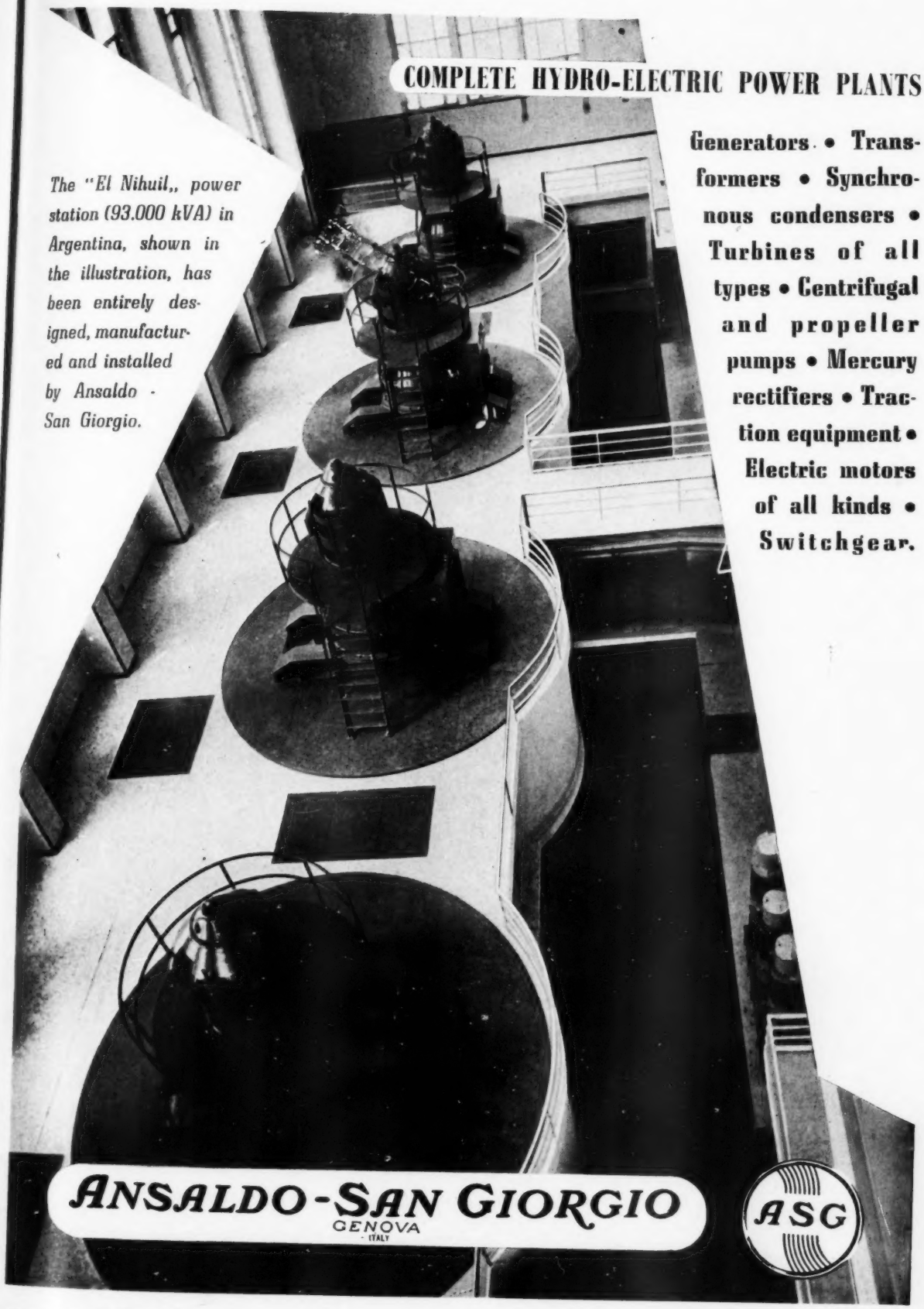

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The "El Nihuil,, power station (93.000 kVA) in Argentina, shown in the illustration, has been entirely designed, manufactured and installed by Ansaldo - San Giorgio.

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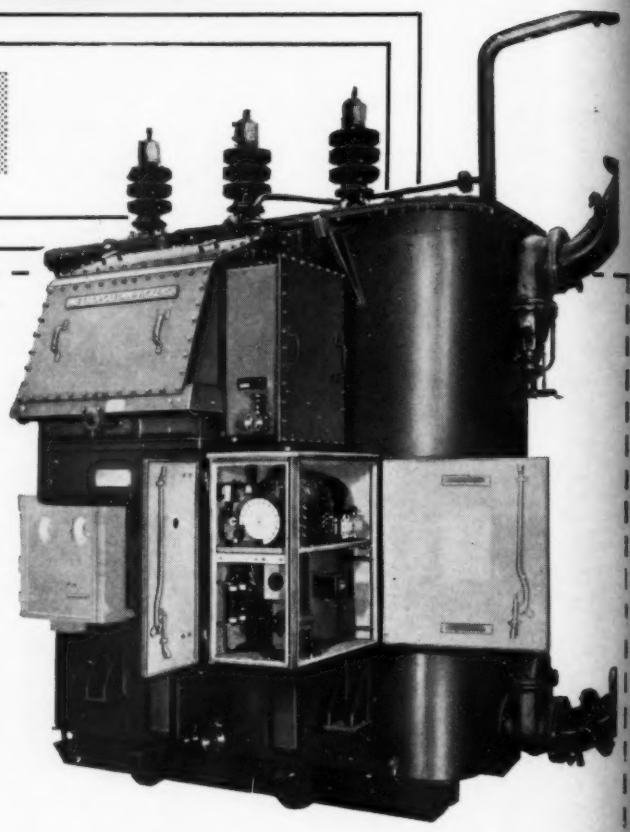
TYPE M



Designed for use on transformers of up to 275 kV, the Type M tap changer has a simple and reliable operating mechanism which ensures completion of tap change. Transition resistors—liberally rated and mechanically robust—minimize arcing and burning at the contacts of the diverter switches, and ease the switching duty. The complete tap changer with its motor driving gear is compactly assembled as a single, iron-clad unit and bolted to a flange on the transformer tank.

Other important features:

- *Any number of steps up to 18 can be accommodated*
- *Once initiated, a tap change cannot be interrupted*
- *Continuity of supply maintained during a tap change*
- *Separate drop-down tank allows convenient inspection of diverter switches*
- *Diverter switches are rapid in operation*
- *Contact bounce at diverter switches eliminated*
- *Minimum voltage variation during a tap change*
- *Moving contacts have single bolt fixing and are accessible from the front.*

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